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THE FLOW OF WATER IN FLUMES

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INTRODUCTION

The carrying capacity of flumes [2] is discussed in this bulletin. Important divisions of many main canals, used for irrigation, by hydroelectric, mining, or municipal use, perhaps miles long, consist wholly

[1] The writer desires to acknowledge indebtedness to the various engineers and managers of irrigation and hydroelectric systems who permitted and aided in tests upon the flumes in their charge. Acknowledgment is also made to the engineers of the U.S. Bureau of Reclamation who made available a large file of data and field notes pertaining to tests made from time to time by that organization. Where data have been secured from other sources the necessary references are given in the literature citations or in the notes explaining table 2. In experimental tests and computations the writer has been assisted at various times by the late J. M. Brockway, and by P. A. Ewing, K. J. Bermel, I. M. Ingerson, and A. S. Guerard, Jr. He has also drawn extensively upon many discussions with Julian Hinds, J. C. Stevens, and also with A. C. Norton, who has been associated with the construction and operation of flumes throughout the West for more than 20 years.

[2] A "flume" is a complete, self-supporting, artificial water conduit with a free-water surface. Although it is usually a channel body carried on trestle bents, the flume body may rest directly on a bench excavated in a hill or canyon side. Less often, it is partly or wholly backfilled so that it becomes similar to an ordinary canal lining. If the channel material and the backfill are essential to each other for mutual protection, then removal of the inside shell would still leave an unlined canal. If the backfill were removed from the sides of a flume structure, the structure would not be impaired and would still be a flume. A pipe section used as an open channel, carried across a gulch on a trestle, has the characteristics of a flume. The old Roman aqueducts of masonry arch substructures were essentially flumes. On the other hand, some English technical articles refer to short measuring structures as "flumes", but larger structures, that would be known as "flumes" in this country, as "aqueducts." The latter term, in this country, is applied to many major water conduits, especially those several miles long and using many types of conduit, not necessarily including any flume. Likewise reference (*1*) [3] describes a "steel canal" which would be called a steel flume in the United States. In irrigation use, especially in Colorado, the "rating flume" was a short, boxlike structure set in a canal and calibrated with a current meter for making measurements of flow. It had some of the characteristics of a very short flume as discussed herein. From a capacity standpoint, the metal flume, the wood-plank flume, and the stave flume are not easily confused with any other type of conduit. The concrete flume, however, is not essentially different in capacity features from a concrete-lined canal except that the walls are usually vertical and the velocities are usually higher than in lined canals. Many concrete chute structures are essentially of flume construction but are backed with earth almost or quite to the top of the walls. Wooden or metal chute flumes are usually laid on sills or short posts in the open.

[3] Italic numbers in parentheses refer to Literature Cited, p. 94.

of flume structures. In point of numbers, however, most flumes are relatively short structures spanning gulches, streams, or other depressions between sections of open canal. Being usually a high-velocity structure with V (see notation, p. 5) ranging from about 3 to 15 feet per second, the kinetic effects are more in evidence than in the ordinary canal with velocities of from 2 to 4 feet. Where the flume is a major structure of great length, the problem of capacity is determined as for any other open channel, assuming that uniform flow will be developed at normal depth through the greater part of the flume length. Here the inlet loss and conversion of static head of elevation to kinetic velocity head as the water is accelerated is usually a relatively minor item compared to the total loss. So, too, is the recovery of head of elevation as the water is decelerated at the outlet of the flume. The head lost in overcoming frictional resistances far overshadows the changes along the hydraulic gradient due to inlet and outlet conditions.

On the other hand, most flumes are so short that normal flow does not become established. The frictional resistances cause unimportant changes in the hydraulic gradient and the constructed slope of the flume could be altered quite appreciably without general effect on the flow through the structure; but the water stage in the outlet pool has a material effect on the flow through the flume and nonuniform flow is the rule rather than the exception. Furthermore, the relationship between energy content and normal depth is of much more moment in a short structure than in a long one.

Historically, the flume dates from the Roman aqueducts (*23*). For crossing wide, deep depressions these offered the only solution available in their time. Today similar problems would be studied with alternate possibilities: an open-channel conduit in the form of a flume, an inverted siphon pipe line of concrete, steel or wood staves, or even gravity flow down the near side of the depression with a pumping lift up the far side. For either the siphon or the pumping line, the Romans had no materials of construction in units of magnitude and strength to withstand the static pressures that would be developed at the deeper parts of the depressions.

Linking these aqueducts with the last few centuries, similar elevated masonry flumes are found in France, Spain, Africa, and Mexico. The oldest flume in the United States that has been called to the attention of the writer carries the Espada Ditch irrigation supply across Piedras Creek near San Antonio, Tex. It was built by the Spaniards, about 1731, of rough rock masonry in the form of a 2-arched bridge, surmounted by the flume proper (*17, p. 45*).

About 1850 irrigation in Utah and hydraulic mining in California led to the development of the box flume, a rectangular, wood-plank structure with battens over the cracks to reduce leakage (fig. 1).

For the next half century this was practically the only type of construction used for flumes in this country. At the first convention of American Society of Irrigation Engineers in 1891, extended discussion was given to this type of flume (*28*).

From the late eighties on through the first decade of the present century, the standard flumes of today had their beginnings. The natural development of the plank flume is exemplified in the standard construction of structures in Provo and Logan canyons, Utah (pl. 3, B and fig. 1). The battens were omitted and tightness secured by inserting splines between the plank edges. The full width of each surfaced plank is available.

For the old San Diego flume, redwood staves were used with semicircular bottom, and side walls tangent at the mid-diameter. A few stave flumes of this shape were constructed with several forms of cradles, but for various reasons the type was practically abandoned until some 15 years ago when the shape was changed to its present form—about five eighths of a circle—and banded like a stave pipe with spreader caps to take the place of the upper portion of the circle (pl. 3, C and fig. 7). Metal plates were used to form the rectangular flumes of the Bear River Canal in Utah, and of the Henares Canal in

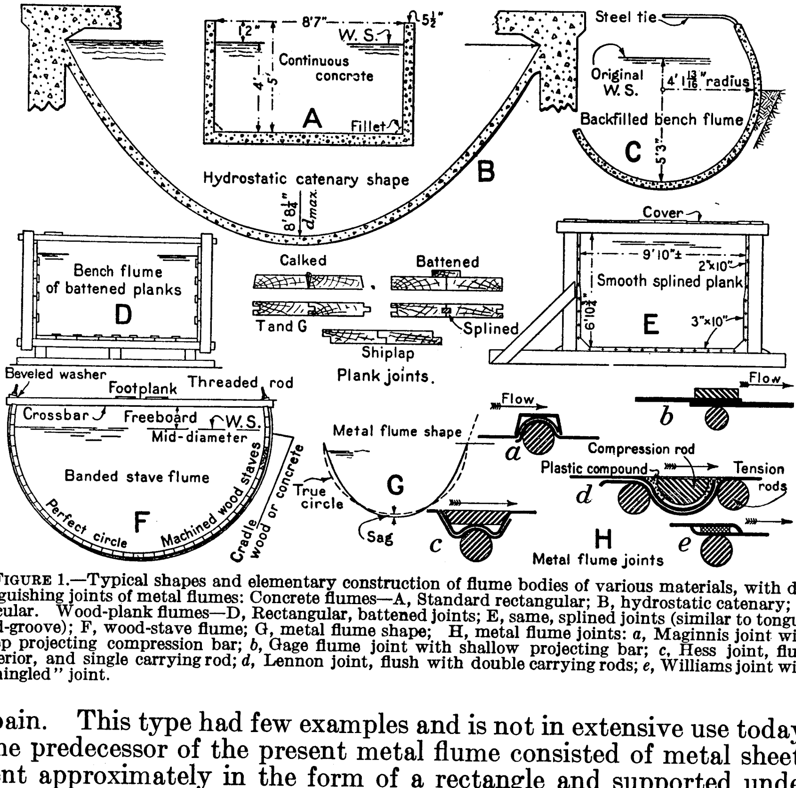

FIGURE 1.—Typical shapes and elementary construction of flume bodies of various materials, with distinguishing joints of metal flumes: Concrete flumes—A, Standard rectangular; B, hydrostatic catenary; C, circular. Wood-plank flumes—D, Rectangular, battened joints; E, same, splined joints (similar to tongue-and-groove); F, wood-stave flume; G, metal flume shape; H, metal flume joints: *a*, Maginnis joint with deep projecting compression bar; *b*, Gage flume joint with shallow projecting bar; *c*, Hess joint, flush interior, and single carrying rod; *d*, Lennon joint, flush with double carrying rods; *e*, Williams joint with "shingled" joint.

Spain. This type had few examples and is not in extensive use today. The predecessor of the present metal flume consisted of metal sheets bent approximately in the form of a rectangle and supported under the bottom and sides. Some of the early stave flumes were suspended from a girder along the top edge by carrier rods. The sheet-metal flume is now invariably suspended by rods, and the stave flume is supported by cradles.

About 1908, Patrick Maginnis of Kimball, Nebr., was making sheet-metal feeding troughs for sheep. At a farmer's suggestion, a flume was developed by placing several of these troughs end to end. Except for the smallest sizes, the flexible sheets, laid transversely, were suspended from timber girders along the top edge by means of carrier rods, which also formed the joint at the edges of the sheets. If perfectly flexible—made of canvas, for example—a trough, loaded with water, would take the shape of the hydrostatic catenary, with internal pressures at all points normal to the curve and proportional to the depth below the surface, and with tension the only stress in the sheet. In the earlier flumes of this type, the interior compression bands between metal sheets were of rectangular section and projected into the water prism (fig. 1, H, *a*).

Hundreds of such flumes were used on projects constructed from 1910 to 1915. It was some time before experimental data indicated that the value of Kutter's n for this flume ranged from 0.016 to 0.018 or more. Thus, the capacities were found to be some 25 percent less than the early estimates upon which many of these flumes were constructed. A similar flume with a wide band of thinner section is still manufactured (fig. 1, H, *b*). In 1911, G. L. Hess, and later William Lennon (fig. 1, H, *c* and *d*), removed the excessive obstruction to flow by countersinking the compression member until it was approximately flush with the interior surface of the flume sheets. Prior to the present Lennon type of continuous-interior flume, this joint was used for flumes of corrugated metal similar to the well-known corrugated-iron road culverts. Flumes of this type were more rigid than those of the smoother types, but the carrying capacity was so much reduced that they gradually fell into disuse. The measurements listed in table 2 show the high retardation factors involved.

In its latest development the metal flume is rolled in a shape approximating the hydrostatic catenary. The true shape could be adopted for any one loading of water, but this changes with each depth of flow. The maximum sag occurs when the flume is less than half full. Consequently, a compromise curve was adopted that appears to satisfy operative conditions with varying flows. For many years, these flumes were rarely painted and the smooth, metal surface gave a high capacity unless the sheet joints opened enough to injure the smooth interior, or the lack of sufficient carrying rods allowed each sheet to "scallop." In either case, of course, the capacity was reduced. At present, it is customary to use rods enough to prevent scalloping and to paint the interiors of metal flumes every few years. The best of paint work appears to offer but little more obstruction to flow than did the clean metal, but the type and manner of application of the paint may reduce the capacity 10 percent or more.

The largest of the masonry flumes on irrigation systems date from the middle of the nineteenth century. They were developed by the English engineers to convey the great canals of India over major streams in "aqueducts" or to carry the streams over the canals in "superpassages." Essentially, both are flumes in our parlance. The twin-structure Solani Aqueduct, the magnificent Nadrai Aqueduct and the superpassage to carry some 15,000 second-feet of water in the Puthri Torrent are well known to students of irrigation in India. All of these would class as "short" structures, with the effective slope largely determined by water stages upstream and downstream, rather than by the constructed slope of the floor.

The reinforced concrete flume is a development of the twentieth century. A flume of this type may be made in many different ways, each having its own characteristic interior surfaces and material differences in capacity. It may be poured in place, in panels from 10 to 20 feet long with expansion joints between the panels. Sometimes the side units are precast during the irrigation season and set up after water is turned out of the canal in the fall of the year (pl. 1, B). The Hilts Sag flume of the South San Joaquin irrigation district in California replaced an old box flume. First the concrete trestle structure was built under the wooden flume. Later the wooden box was replaced with precast floor and side walls (pl. 15, B). Gunite has been shot from the outside against smooth wooden forms and shot directly in constructing the floor (*2*). The inner surface of the side walls needs no further treatment. The floor can be much improved by using a trowel or other smoothing device (pl. 1, C). Gunite shot from the inside yields a very rough interior unless it is smoothed afterward (pl. 12, A). The concrete flume is usually of rectangular form, but has been built in circular shape in both poured forms and precast units (pl. 1, A and 7, B) and has also been poured in the form of the hydrostatic catenary, thus eliminating all bending moment and sheer in the reinforcing steel under maximum load (pl. 8, B) (*20*).

The flume is nearly always an expensive structure per unit of length in comparison with the cost of canals. Size, therefore, is of great importance from the economic as well as from the hydraulic standpoint. With the materials of construction available, limiting velocities are very high. For a given maximum flow, Q, area of water prism is inversely proportional to velocity. All the factors indicate that the limits of size and slope cover a range far in excess of that practicable for many types of conduits. Flume velocities are usually from 4 to 8 feet per second; many are from 8 to 12 or 15 feet per second; and flume chutes are commonly found with velocities from 20 to 30 feet per second, with isolated cases up to 80 or more. Irrigation head flumes delivering water from a lateral to furrows in a field are but a few inches in width. Lateral and main canal flumes are commonly 6 to 10 feet wide with water 2 to 6 feet deep. Large flumes are common in widths of 12 to 20 feet or more, while flume aqueducts in India range up to some 200 feet in width and flume superpassages to 400 feet in width.

Most irrigation or hydroelectric systems have some flumes. An idea of their importance in numbers can be found in a recent report (*11*) which states that the projects constructed by the United States Bureau of Reclamation have (1930) 5,260 flumes of concrete, metal, and wood, aggregating nearly 900,000 linear feet, 62 percent of which is wood flume.

Yet the census of 1929 indicates the area irrigated on the Federal projects is less than 8 percent of the total. Although it is certain that this ratio of fluming to area would not be applicable to the entire irrigated area, the total number and mileage of flume is great. The number and aggregate length of the additional flumes used on hydroelectric, municipal, and mining systems would probably equal those in irrigation use.

NOTATION AND NOMENCLATURE [4]

Notation.—Throughout this bulletin, the following symbols will be used to designate the same element. Additional description and definition to be found in Nomenclature, indicated by (see).

A. The weighted mean area of the water cross section throughout the length of reach, considered, in square feet. (See weighted mean.) Also used for area in the abstract.

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[4] The lettered symbols used in this bulletin to express various elements will be given under Notation and explanation of special names and phrases under Nomenclature.

a. The area of any particular water cross section. Usually given a subscript to identify the location and the corresponding elements.

C. Coefficient in Chezy formula, $V=C\sqrt{RS}$.

c. A coefficient or constant.

c_e. Coefficient of entry to apply to the increase in velocity heads.

c_o. Coefficient of outlet to apply to the decrease in velocity heads.

D. Diameter of semicircular flume, in feet.

d. Depth of water in the channel, in feet, usually given a subscript to identify location as d_1, d_2, etc.

d_c. Critical depth (see).

d_n. Normal depth (see).

Δ. Increment, change, as Δh=change in velocity heads; ΔZ change in surface elevation.

E. The elevation of a point on the energy line, in feet. Usually given a subscript to identify location. $E=Z+h=k+d+h$. $E_1-E_2=h_f$.

g. The gravitational constant=32.2 in English measures.

H. Energy content$=d+h$. Bakhmeteff's "specific energy" (*4*).

H_{min}. Minimum energy content$=d_c+h_c$.

h. Velocity head, assumed$=v^2/2g$,[5] in feet. The drop in elevation of the water surface (W. S.) necessary to generate the velocity under consideration.

h_c. Velocity head for Belangér's critical velocity, v_c, in feet.

h_e. Entry loss in feet. The fall in the energy line between entrance to flume transition and entrance to flume proper.

h_f. Friction loss. The fall in the energy line, through the length of reach considered, in feet. The difference in the values of *E* at the two ends of the reach. For idealized steady uniform flow only, the fall in the energy line, in the water surface and in the channel bed are alike. The loss due to all hydraulic friction rather than perimeter-contact friction only.

h_o. Outlet loss in feet. The fall in the energy line between the outlet proper of the flume and the end of the outlet transition or the point of maximum recovery of velocity head.

h_r. Recovered velocity head. The kinetic energy recovered and converted to head of elevation while h_o is developing at the outlet. Expressed as a percentage of difference in velocity heads for flume section at outlet proper and canal section at greatest elevation of *Z*. Also expressed in feet as *Z*(canal)—*Z*(flume outlet).

i. Head lost by impact and eddies, as in the hydraulic jump.

J. Height of hydraulic jump expressed as ratio of depth after, to depth before the jump. $d+h=d'+h'+i$, *d* and *h* referring to elements before the jump, and d' and h' to elements after the jump.

K. Bakhmeteff's symbol of conveyance of a channel$=AC\sqrt{R}$ (*4, p. 13*).

k. Elevation of flume bed or canal bed above datum, in feet.

L. Length of reach considered, measured along the bed slope, in feet.

M. Momentum$=Q^2/Ag$ in cubic units of water. (See p. 19.)

M'. Bakhmeteff's M'-function$=A\sqrt{AT}$. (See p. 81.)

λ. Bakhmeteff's "Kinetic flow factor"$=\frac{Q^2}{A^2gd}=\frac{2h}{d}$. (See p. 84.)

n. Coefficient of hydraulic friction in Kutter's formula.

n'. Coefficient of hydraulic friction in Manning's formula.

n. A subscript to denote elements at normal flow, such as v_n, d_n.

P. Wetted perimeter in the abstract. Also hydrostatic pressure in cubic units of water.

p. Wetted perimeter, in feet, at a particular cross section, from which $a/p=r$.

Q. The amount of flow, in cubic feet per second, under consideration. Design-Q', the flow used as maximum capacity in the design of a flume or other structure. Q_c, critical flow, at v_c and at depth d_c.

R. Mean value of hydraulic radius r_1, etc. through reach considered, in feet, for computation of friction factors. Should not be taken as A/P where *P* would be the mean value of the wetted perimeters p_1, etc. Also *R* used for hydraulic radius in the abstract.

r. Hydraulic radius at a particular cross section, in feet, usually given a subscript to identify location as $r_1=a_1/p_1$.

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[5] This is the usual interpretation of velocity head. Strictly speaking it should be the mean value of the velocity heads for all the elements of flow across the section, rather than the velocity head for the mean velocity across the section. The true value may be 15 percent or more in excess of *h* as given here. Muckelston, discussing Hinds (*14, p. 1,461*) suggests adding 4 percent to the value of *h* as above, in order to approximate the true velocity head.

S. The slope of the energy line (*E* line); always downward in direction of flow. Never rises. The slope factor in flow formulas such as Kutter's. $S=\frac{h_f}{L}$ in feet per foot $=\frac{E_1-E_2}{L}\cdot$ *S* is not the slope of the water surface. In idealized uniform flow, it is parallel to and hence equal to that of the surface and that of the channel bed. In flume hydraulics the surface falls rapidly above or at the inlet of a flume; rises and falls in the flume, and usually rises rapidly immediately beyond the flume outlet.

s. Slope of the bed of the flume; usually downward. May be horizontal for very short flumes. May rise or fall gently or abruptly at either or both ends of the flume. The flume bed may be well above or well below the level of the canal bed at the ends of the flume. In design, *s* usually computed as parallel to *E line* and to water surface for an assumed normal surface at capacity flow.

T. Width of water surface at section under consideration, in feet.

V. The mean velocity of the water through the reach under consideration, in feet per second. Approximates Q/A if cross sections are taken often enough so that mean area *A* closely approximates the mean value of a great many local areas (a_1 . . . etc.). For many cross sections, best approximation is mean weighted value of v_1, v_a, v_b . . . v_2. In flume chutes, the swell of the water due to entrainment of air annuls the continuity equation $V=Q/A$. (See p. 90.) *V* will also be used for velocity in the abstract.

v. The average velocity of water across a local cross section, usually given a subscript to identify location, as $v_1=Q/a_1$.

Z. Elevation of the water surface above datum, in feet, usually given a subscript to identify location as Z_1, Z_2, etc.

W.S. Water surface; usually sloping downward. May remain quite level though flowing rapidly throughout length of flume, say 1,000 feet long. This emphasizes fact that energy slope, *S*, is effective, not water slope. Usually rises appreciably, in recovery of velocity head, beyond lower end of flume. May rise abruptly, through the hydraulic jump, either within the flume proper or at the outlet. May rise and fall intermittently—above and below critical depth, either with or without the jump, for flow near critical depth in a uniform channel, such as a flume.

Nomenclature—In this bulletin certain words and phrases have special meanings not ordinarily found in the dictionary.

Canal. The channel leading up to and away from the flume.

Control. Where conditions are such that critical flow is developed at a section of the conduit, then this location is called a "control", because the flow upstream is independent of the water stage below the control. If the conduit or pool velocity above the control is slower than the critical velocity, maximum flow with minimum energy content ($H_{\text{min.}}$) holds at the control. If the conditions above the control location change to cause velocity faster than the critical, the control passes from this location to some other location farther upstream where critical flow is developed. A definite control section for all flows offers an excellent place from which to compute surface curves.

Critical depth. In flume design and operation critical velocity and the depth at which this velocity holds are of importance. For any given section, quantity of flow, *Q*, and the total energy, $H>d_c+h_c$, there are two depths of water for which $d+h$ are identical. These are called the "alternate stages" (pl. 12, C). At critical depth, these two stages merge (fig. 5, and pl. 14, B and C). For any other such value of *H*, there are two other depths that are also conjugate. This rather complex condition is made clearer by a study of the *H*, *d* curve in figure 7. Water at less than critical depth is flowing at shooting velocities and the flume usually becomes a "chute." If the depth is greater than the critical then we have streaming velocities, subject to both the backwater curve and the drop-down curve. Shooting velocities are not subject to long backwater or drop-down curves, as ordinarily considered. They do exist under the condition of accelerating flow, from critical velocity at the top of a chute to a much faster velocity that is normal for the chute under consideration. However, the writer believes this surface curve to be similar to that of the entering nappe at the inlet of a smooth-transitioned flume (pl. 5, A).

Energy gradient. Strictly, the rate of fall of the energy line, equal to the energy slope $S=\frac{h_f}{L}=\frac{E_1-E_2}{L}\cdot$ By analogy, the energy line itself.

Energy line. The energy grade line. The locus of Bernoulli's summation, considering losses; hence, the E line. Not to be confused with water surface. For tapered flow the energy line is taken as straight for the reach considered. If the taper is caused by checked water, then the depth increases, velocity decreases and the E line is slightly concave upward (fig. 2, C). If the velocity increases the E line is slightly convex upward (fig. 2, B).

Energy content. Energy content curve is a H, d curve in figure 7 for a given Q.

Free flow. As applied to flume outlets; requires a stage of water in the canal below (tail water) that permits flow at the outlet to be at critical depth (see). This condition may be attained with a recovery of velocity head so that the tail-water stage is well above the stage in the flume outlet, $Z = d_c + h_r$. Any stage of tail water below this does not change the stage of flow in the flume outlet; causes a definite break in the energy gradient; and acts as a drop, and the flow from the flume is said to be free.

Flume. An artificial open conduit of concrete, metal, or wood on a prepared grade, trestle, or bridge. A flume is a complete structure for the conveyance of a flow of water.

- Bench flume. A flume bedded down on a flat shelf or bench excavated on a definite grade along canyon or mountain sides. Some have been partially or wholly backfilled and so confused with canal linings. Some are covered but the water conveyed is not under pressure. (See Trestle flume.)
- Long flume. A flume of such length that at least through a portion of it the water flows at approximately normal depths for all values of Q. Back-water or drop-off curves may influence the surface and velocity toward the outlet end. These elements can be modified in many ways at the inlet end. The controlling capacity elements are determined by a solution for uniform flow at any normal depth and this controlling capacity can not be improved by any changes in water stage at the outlet end as can a short flume.
- Short flume. A flume of such length that the water surface curves are determined throughout by the water stages in the canal above and below, or by a control developed at the inlet or outlet ends of the flume. In a short flume normal flow develops by coincidence only. However, short flumes as well as long flumes are usually designed for an idealized uniform flow.
- Trestle flume. A flume body on a definite grade carried on trestle bents of varying heights above surrounding terrain. English and Canadian engineers often call them aqueducts when of imposing dimensions. The old Roman aqueducts were essentially covered flumes. Many conduits called aqueducts in the United States have little or no fluming throughout their lengths.

Hydraulic friction. Cause of loss of head in flowing water; includes influence of channel surface and alignment, eddy, impact, and other losses besides friction with the containing channel; excludes enlargement, contraction, and "special losses."

Normal depth. The depth of water at normal flow d_n (see). The idealized depth, resulting from computations for uniform flow. Some writers prefer neutral depth to avoid any confusion of normal as a synonym for "at right angles to." For flow down steep chutes, depth is usually measured normal (at right angles) to the slope of the chute rather than vertically. Only in this connection can confusion arise in the use of normal depth as defined above.

Normal flow. Uniform flow in a uniform channel, satisfying the solution for a flow formula, such as Kutter's. Under this condition, the bed slope, the water surface, and the energy line are parallel. Though useful in design, such uniform flow is seldom found in field experiments. A perusal of table 3 indicates the general tendency for tapered flow to exist. Long, straight channels of uniform shape and uniform surface would develop this idealized flow. It should not be taken for granted in any field tests for values of n. Some writers prefer neutral flow to normal flow.

Shooting and streaming flows. (See Critical depth.)

Weighted values. Throughout this bulletin values of local elements, such as a, r, etc., will be weighted in the determination of corresponding mean values, A, R, etc., in accordance with the length of reach each local element influences.

TYPES OF FLUME

Flume structures are built of several materials and usually are of distinctive shape as shown in figure 1. However, all of the characteristic materials are used for flumes of distinct types, which may be divided in two ways from the flow standpoint: Uniform or nonuniform flow; streaming flow or shooting flow. These groups may be subdivided according to type, as follows:

Type A, long flumes, including some steady, uniform flow.

Type B, short flumes, with nonuniform flow in a uniform channel.

Type C, relatively low-velocity flumes for flows slower than the critical.

Type D, relatively high-velocity flumes for flows faster than the critical, commonly called "chutes". Often with complex flow due to changes in constructed gradient.

The flow in types A and B must be in combination with types C or D. Sometimes both C and D are found in a single structure of either A or B type. The writer estimates the proportions of the combinations of types about as follows: Type A-C, 20 percent; type B-C, 75 percent; type A-D, 2 percent; and type B-D, 3 percent. In connection with types C and D, the word "relatively" is used because shooting flow may occur in a small flume at a velocity of 3 or 4 feet per second; but in a large flume this stage is deferred until the velocity is much higher.

FORMULAS FOR FLOW OF WATER IN FLUMES

Long flumes, in which the anticipated velocities are slower than the critical, can be designed like any other open channels, using one of the standard formulas for uniform flow. However, the high velocities usually associated with flumes must be kept in mind. Definite provision must be made for the drop in water surface necessary to develop the high velocity as water enters the flume from the canal or other conduit at a lower velocity. Greater freeboard is required to care for anticipated or possible waves. At the outlet transition, the recovery of velocity head should be considered.

The determination of the actual water prism with its hydraulic elements is the same as for any open channel. In the United States, the Kutter formula, as elaborated from the simple relation as given by Chezy, is still in general use. Values of n in the Kutter formula and of n' in the Manning formula are quite alike through the range usually associated with flume flow; hence, the Manning formula and diagrams based thereon can be used to obtain approximately the same results as those obtained by the Kutter formula. The two formulas do not yield approximately identical results in the higher values of n, say from 0.030 upward. However, the proper coefficient of roughness to use in any formula remains a matter of judgment, based upon empirical data obtained from flumes similar in material and conditions to the one under consideration. The empirical data listed in this bulletin give the field measurements and supplementary office computations finally resulting in values of the retardation or flow coefficients for all formulas of the usual type, which neglect any influence of viscosity except as such influence existed in the original field tests upon which most formulas were based.

Short flumes are usually designed for the desired maximum capacity in much the same way as long flumes, except where the structure is to act as a drop as well as a flume. However, they are more elastic in their hydraulic properties than a long flume. Even if designed for uniform flow at a normal depth with the desired freeboard, the flow, if inadequate, may be increased by means that would not affect a long flume. The shorter the flume, the more is its maximum capacity dependent on its place in the vertical plane with regard to the water prism and energy line in the canal at either end. Likewise, the shorter the flume, the less is its capacity dependent on its constructed slope. A very short flume—crossing side drainage or a narrow depression or replacing a washed-out section of canal—may have a very steep slope, or a level bottom, or even a bottom that slopes upward in the direction of flow; and the full range in capacity for these conditions may be less than the range as influenced by its position in the scheme of levels. (See pl. 14, C.) Sometimes strange paradoxes result from extended study or trial with short flumes. Several years ago a short section of a long metal flume was washed out. The canal company replaced the missing section of approximately circular shape with a short piece of rectangular box flume, having the same gradient that the metal flume had. Using a higher value of Kutter's *n* for the rough board flume, the design resulted in a relatively large section for the box flume. This section became the bottle-neck in the whole conduit. It would not carry the desired flow, not because it was too small, but because it was too large. The losses in the conduit enlargement at the section entrance and the contraction at its outlet, first rapidly decreasing the velocity and then as rapidly increasing it, completely overshadowed the negligible difference in friction loss due to difference in conduit material. Had cradles approximating the shape of the metal flume been lined with the boards used for the box flume, the resulting replacement would have worked as desired, but perhaps encroaching a little on the freeboard upstream in order to overcome the slight difference in friction loss.

The coefficient of retardation has been computed for three of the best known formulas used in the United States in designing open channels. The formulas considered are the Chezy formula, (1); the Kutter formula, (2); and the Manning formula, (3).

In 1775 Chezy, a French engineer, advanced the following formula for calculating the flow of water in open channels:

$$V = C\sqrt{RS} \tag{1}$$

This formula was based on the assumption that the velocity of water flowing in a long, uniform channel does not increase for each succeeding second of its passage as would be the case if it followed, unhindered, the law of gravity; but that it acquires a certain velocity early in its flow, and from that time the velocity remains quite constant as long as the surrounding conditions are not changed, the tendency for the velocity to increase being just counteracted by the various retarding influences. The conditions upon which the assumption is based are approximated in long, straight flumes.

The coefficient C was supposed to care for all of the various factors affecting the velocity, such as friction between the moving filaments of water and the containing channels, but it did not involve the slope and the mean hydraulic radius. After some three quarters of a century of use, it was found that C was not constant but a rather complicated variable.

Substituting this variable in formula (1) we have Kutter's formula, expressed in English measures:

$$V=\left(\frac{\frac{1.811}{n}+41.66+\frac{0.00281}{S}}{1+\left\{41.66+\frac{0.00281}{S}\right\}\frac{n}{\sqrt{R}}}\right)\sqrt{RS} \tag{2}$$

This formula takes into consideration the influences of hydraulic grade and of the mean hydraulic radius upon the coefficient C, and introduces a new variable, n, which is supposed to represent all the retarding influences.

In this elaborated form the above formula represents a vast amount of mathematical plotting and deduction by Kutter and Ganguillet (*10*), engineers, in Berne, Switzerland. It was developed in 1869 from the data covering 81 different gagings of rivers and canals, ranging from channels a few inches wide to the Mississippi River.

Although there has been much protesting against the Kutter formula, particularly respecting the term including S, it is still the mainstay of the irrigation engineer in the design of open channels. The courts of the West, in particular, look askance at any attempt to establish the capacity of an open channel by any formula other than Kutter's. In Europe the Kutter formula is used in some countries, while others use the Manning, Bazin, or other later formulas.

In an effort to modify the complex form of Kutter's formula, the Manning formula appeared some 40 years ago and recently its general acceptance has been urged in the United States. As material for use with this formula, values of n' are given with Kutter's n, in table 2.

The values of n' in the Manning formula (3), are sufficiently close to the values of n in the Kutter formula so that the same values may be used by engineers partial to the Manning formula, at least through the range usually found in flumes. The chief advantage of the latter is its simplicity, but as the Kutter formula is seldom computed—diagrams and tables being quite generally used—this objection to the Kutter formula is not material. The Manning formula is

$$V=\frac{1.486}{n'}R^{0.67}S^{0.50} \tag{3}$$

The flow formulas given above provide the hydraulic elements for a flume of given or assumed shape and dimensions, provided it is long enough for normal flow to develop. In all these formulas, the following may be noted: The area of the water prism, A, the wetted perimeter, P, the hydraulic radius, R, the length, L, the friction loss, h_f and the slope, S, are dimensional and assuredly computed. The retardation factor assumed in the computations is a matter of judgment, based on such empirical data as those listed in this publication, interpreted by the designer himself or as detailed in recommendations of the writer (p. 51).

VELOCITY HEAD

The mean velocity of the water, V, a direct factor in the quantity of flow, is the velocity that will be maintained, without acceleration or deceleration, if the assumptions in the flow formula are correct. But that formula does not provide for the generation of this velocity in the first place. In the past this provision has often been overlooked. It is called the velocity head, symbolized by h. It is absolutely essential in flume structures. The velocity head has been assumed to be the same as for the falling body in ordinary computations.

The formula for the velocity of a falling body, $V=\sqrt{2gh}$, transposed and expanded becomes

$$\text{Velocity head}=h=\frac{V^2}{2g} \qquad (4)$$

The essential nature of h in flume problems may be clarified by an example: If the mean velocity of the water in a flume is 8 feet per second, the surface of the water at the beginning of this rate of flow must be at least $\frac{8^2}{2g}=\frac{64}{64.4}=0.995$ feet (say 1 foot) below the energy line E, at that point. If the water upstream from the point is pooled, that is, without appreciable velocity, the water surface and E are at the same elevation. If the water upstream, in an earth canal, for example, has a velocity v_0, its velocity head, equal to $\frac{v^2_0}{2g}$, must be added to the elevation of the water surface, Z_0, to get the point on the energy line, E_0. As a general formula

$$E=Z+\frac{v^2}{2g}=Z+h \qquad (5)$$

and

$$\text{for still water, since } v=0.00,\ E=Z \qquad (6)$$

Therefore, for either a long or a short flume, velocity head for the increased velocity at the upper end of the flume takes the form of a surface drop, shown in many of the views in this bulletin. (See pls. 4 and 5, and fig. 2.)

ENTRY LOSS

The loss of head at entry is shown in figure 2 by the slight, steep drop in the energy line and not by the deep plunge of the water surface. The latter has often been mistaken for entry loss. It is merely the conversion of head of elevation to velocity head described in the last few paragraphs above. It can be understood as an investment and becomes a loss only as measured by lack of recovery further down the flume or at the outlet.

Discussion in terms of the energy line and energy head is new to many readers who will recognize the older form which amounts to the same thing:

$$\Delta Z=Z_0-Z_1=\Delta h+c_e\Delta h=\Delta h(1+c_e)=\frac{v^2_1-v^2_0}{2g}(1+c_e) \qquad (7)$$

The drop in water surface, ΔZ, is equal to the difference in velocity heads for the head water in the forebay or leading canal and for the water in the flume at the upper end, plus a small friction loss, plus the entry loss. In the United States, this is usually taken as a constant, multiplied by the difference in velocity heads, or $c_e\Delta h$. Some authorities apply c_e to the higher velocity involved. Generally, the difference, due to method, is not of moment since the lower velocity usually has a much smaller head than the higher one.

The coefficient of entry-loss, c_e,[6] including friction in the inlet transition, may be taken as:

[6] The suggested values of c_e may be understood as conservative conclusions from all data on entry losses now available, with friction included.

(1) 0.20 for square-ended bulkhead entrances to flumes (pl. 4, A).
(2) 0.15 for entrance wings set 30° to 45° to the axis of the flume (pl. 4, B).
(3) 0.10 for "cylinder quadrant" inlets: vertical, circular, wings (pls. 5 and 8, A).

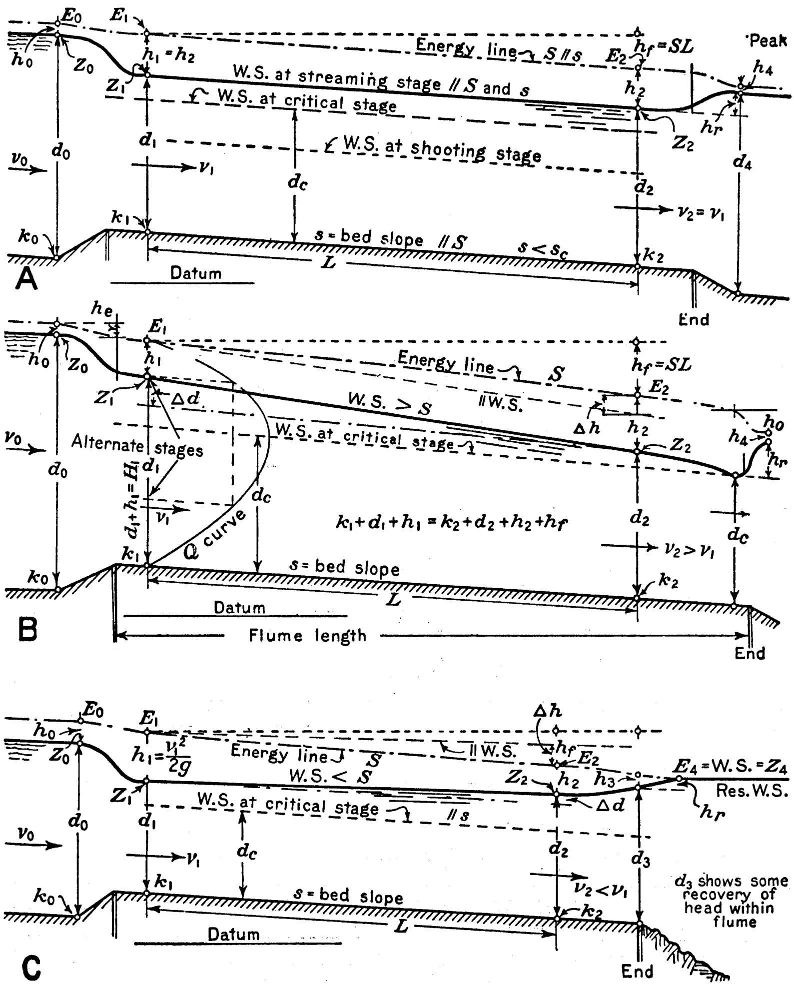


FIGURE 2.—Diagram of three principal flow conditions in flumes: A, Idealized normal flow as usually assumed in design and approximated in actual flow of long flumes; B, accelerating flow, found in short flumes and at lower ends of long flumes where the tail-water stage is relatively low; C, decelerating flow, found in short flumes and at lower ends of long flumes where tail-water stage is relatively high.

(4) 0.10 for warped transition inlets following computations as given on page 73 (pl. 4, C).

The values in (3) and (4), above, indicate that there is probably but little inlet loss other than friction under conditions as given. The coefficient influence comes within the zone of error in the assumptions as to velocity heads.

Empirical values of c_e for cylinder quadrant inlets,[7] described on p. 70, listed in table 1 are computed by the formula:

$$c_e=\frac{\Delta Z-\Delta h}{\Delta h}=\frac{Z_0-Z_1+h_0-h_1}{h_1-h_0} \tag{8}$$

in which c_e is the coefficient of entry loss, including friction loss.

TABLE 1.—*Elements of results; experiments on entry loss in cylinder-quadrant transitions*

Reference no., see table 2	Flow Q	Velocity increases, canal to flume v_o v_1	Velocity heads: Canal h_0	Velocity heads: Flume h_1	Velocity heads: Difference Δh	Surface drop: Elevation ΔZ	Surface drop: Proportion $\Delta Z/\Delta h$	Entry coefficient c_e
	Sec.-ft.	*Ft. per sec.*	*Feet*	*Feet*	*Feet*	*Feet*		*Feet*
I.M.I. 15	2.07	1.75 to 4.56	0.021	0.323	0.302	0.302	1.000	(2)
I.M.I. 14 [1]	2.25	1.19 to 4.64	.022	.335	.313	.32	1.022	0.022
I.M.I. 7 [1]	2.34	1.193 to 4.82	.022	.361	.339	.34	1.003	.003
155	8.36	.92 to 3.4	.013	.180	.167	.156	--------	(2)
105	212.59	4.29 to 6.08	.286	.575	.289	.334	1.155	.155
104	314.00	1.53 to 5.46	.036	.464	.428	--------	--------	(2)
166	564.00	1.795 to 4.95	.050	.381	.331	.338	1.022	.022
156	603.00	1.95 to 9.08	.059	1.26	1.20	1.332	1.11	.110

[1] From experiments of I. M. Ingerson mentioned below.
[2] No loss.

Where the velocity in the flume at the upper end is close to or exceeds the critical, the lip of the intake structure may become a control with a long draw-down curve upstream. Through the lower reaches of this canal section, the velocity is liable to exceed the permissible velocity for the material forming the bed (*9*). In such cases it is advisable to study the hydraulic notch (*21*) to the end that velocities above the inlet structure may be held within bounds. The notch is effective only for velocities slower than the critical.

OUTLET LOSS AND RECOVERY OF VELOCITY HEAD

At the outlet end of most flumes, the velocity is reduced and the decrement in velocity head is available, at least in part, to elevate the water surface. This condition is to some extent the reverse of the surface drop noted at the inlet end, but this rise of the water surface is not so nearly perfect as the inlet drop. This rise, termed the "recovery of velocity head", is usually less than the change in the velocity heads. At the inlet, the surface must always drop at least the full difference in velocity heads, plus a small entry loss. At the outlet, the recovery in elevation is always less than the full difference in velocity heads and the outlet loss is taken as equal to the unrecovered velocity head.

[7] The cylinder-quadrant inlet was developed independently as part of the experimental work toward a graduation thesis by I. M. Ingerson at the University of California, working under the direction of B. A. Etcheverry and the writer. Before suggesting it in this bulletin, it was offered to several organizations for field use, to give structures for tests on sizes much larger than those feasible in the laboratory work. One (pl. 8, A) was built on a 30-foot radius for Erskine Creek flume, 18.4 feet in diameter (no. 166). The results were so satisfactory that this transition has been used to replace the entry structures on all the flumes of Borel Canal. Since then many irrigation and hydroelectric systems of the West have adopted them as standard construction. In the opinion of the writer, they are sufficient for velocities up to about 8 feet in the flume if built with a short straight section between the curved wings and the flume proper for velocities higher than say 5 feet per second. For still higher velocities he suggests using the warped-inlet transition assuming the surface curve as a reversed parabola, along the lines laid down by Hinds (*14*).

By a process of reasoning similar to that for the inlet drop, the formula for the change in the water surface is found to be

$$\Delta Z = \Delta h - c_o\ \Delta h = \Delta h(1 - c_o) \tag{9}$$

where c_o is the coefficient of outlet loss. However, the rise in the water surface at the outlet is the recovered portion of the velocity head investment, or

$$\Delta Z = h_r \tag{10}$$

$$\text{the percentage of velocity head change recovered} = \frac{100h_r}{\Delta h} \tag{11}$$

and

$$\text{the percentage lost} = \frac{100\,(\Delta h - h_r)}{\Delta h} \tag{12}$$

From formula (9) above

$$c_o = \frac{\Delta h - \Delta Z}{\Delta h} \tag{13}$$

where c_o, the coefficient of outlet loss, and h_r, the velocity head recovered, may be taken as:

(1) $c_o = 0.75$ and $h_r = 0.25\Delta h$ for square-end bulkhead outlets.
(2) $c_o = 0.50$ and $h_r = 0.50\Delta h$ for outlet wings set at an angle of 30° with axis of flume.
(3) $c_o = 0.25$ and $h_r = 0.75\Delta h$ for long-taper or warped outlet wings, following design as given on page 74.

These coefficients are more conservative than available experimental data may warrant, but conditions at outlets are more unstable than at inlets so more conservative figures should be chosen.

Where the velocity at the outlet end of the flume is liable to be faster than the critical, the hydraulic jump is almost unavoidable; that is, $d_n < d_c$. Slight changes in the stage of the tail water will change the position of the jump, up and down the canal, but the flow in the flume is not affected until the jump is backed up into the flume. In this event, the freeboard near the outlet of the flume will sometimes be insufficient and water will flow over the sides. If the jump occurs out in the canal, a study covering several depths of flow should be made to determine the location of the jump for various values of Q in order to determine the necessity for protecting the canal just below the flume from excessive velocity and consequent erosion.

DETERMINATION OF EFFECTIVE SLOPE

Formulas (1) to (3), above, permit solutions for uniform flow in long reaches of open flume. The slope element in all of these is generally defined as the slope of the water surface. Actually, it is the slope of the energy gradient, a line higher than the water surface and always falling in the direction of flow, and expressed as

$$S = \frac{h_f}{L} \tag{14}$$

where h_f is the fall in the energy gradient in a length along the axis of the flume equal to L feet.

In terms of Bernoulli's theorem

$$k_1 + d_1 + h_1 = k_2 + d_2 + h_2 + h_f \tag{15}$$

For uniform flow $d_1=d_2$ and $h_1=h_2$ and formula (15) becomes

$$k_1-k_2=h_f \tag{16}$$

But k_1-k_2 is the fall in the bottom of the flume, and since $d_1=d_2$ the fall in the water surface is equal to that of the bottom. Therefore, in considering uniform flow, the fall in a given distance is the same for the bottom, for the surface, and for the energy line. In the solution of formulas for the retardation factors, as listed in this publication, the slope has always been taken as the gradient of the energy line. (See column 16, table 3.) That is,

$$S=\frac{\left(Z_1+\frac{v^2_1}{2g}\right)-\left(Z_2+\frac{v^2_2}{2g}\right)}{L}=\frac{(Z_1+h_1)-(Z_2+h_2)}{L}=\frac{E_1-E_2}{L}=\frac{h_f}{L} \tag{17}$$

All local variations in area, and hence in velocity between points 1 and 2, were interpreted as without loss of any part of the velocity head changes, as the taper flow is interpreted as having a perfect transition.

BACKWATER AND DROP-DOWN CURVES

For many short flumes, and at the lower ends of many long flumes, the backwater and drop-down curves can be anticipated in design with a definite economy of construction. Even when not anticipated, either one of these surface curves is quite sure to develop. The true backwater curve is characterized by a slowing-up of velocities with consequent recovery of velocity head. Likewise, the slower velocities require and make less freeboard. The partial recovery of velocity head in the flume proper makes less demand for recovery on the outlet transition, and also projects the flow into the canal with less erosive effect. Where the outlet of a flume will always be a free fall, then the flume sides can be lowered as the drop-down curve makes more freeboard available.

In computing backwater curves, it is advisable to use a formula that does not assume a wide, shallow water prism with the formula developed for flow per unit width. In flume flow, the elements for the whole water prism under consideration, not for a narrow strip, should be the basis for computation. Several arrangements of formulas, based on the complete water section, with final results in terms of a length of channel, L, necessary to develop a given difference in water depth, d, have been offered. Two of these, given in references (*16*) and (*19*), are based on the assumption of complete recovery of any velocity head released in the true backwater curve, and, conversely in the case of the drop-down curve, in the utilization for friction of all fall in the water gradient not necessary to the generation of increasing velocities.

A concrete example of the development of the backwater curve is given in table 11. The cumulative lengths of channel (last column) determine successive depths of water, and the rate of loss of head, S, incident to the assumed uniform flow between successive values of H, is based on complete conservation of energy; i.e., complete recovery of velocity head from station to station.

By formula (15) $k_a+d_a+h_a=k_b+d_b+h_b+h_f$; but $k_a-k_b=sL$, and $h_f=SL$; from which $sL-SL=(d_b+h_b)-(d_a+h_a)$;
and since $d_b+h_b=H_b$, $d_a+h_a=H_a$, and $H_b-H_a=\Delta H$,

$$L=\frac{\Delta H}{s-S} \tag{18}$$

where subscripts a and b apply to the upstream and downstream stations, respectively.

The extent of true backwater is not limited by any critical condition. The stage of water at the outlet end of a flume may be any depth above the normal depth, being limited only by the extent of available freeboard. The drop-down curve, however, has limitations, as is shown in the following paragraphs:

With the discharge at the outlet end approaching a stage lower than that of normal depth, the velocity increases as the sectional areas become smaller; hence the energy slope, S, required for a velocity faster than normal exceeds the bed slope, s, which is equal to the energy slope necessary for maintaining normal velocity only. Therefore formula 18 is revised to read:

$$L=\frac{\Delta H}{S-s} \tag{18a}$$

For both curves, if ΔH is regarded as a positive change in the energy content, the related formulas 18 and 18a can be used to develop the backwater or drop-down curves, respectively, beginning at the downstream end of the flume.

At the outlet end of a flume with bed slope, s, such that normal velocity is slower than critical velocity, the minimum depth that will develop by the drop-down curve is the depth for critical flow; that is, $d_{min.}=d_c$. This condition is clearly shown in plate 10, C.

For any important flume structure, it is suggested that the drop-down curve for capacity flow be developed from normal depth down to a possible brink at critical depth. The total length of the curve will be the limit for a short flume under accelerating-flow conditions. Any excess length places the flume as a long structure.

Likewise, it is suggested that the curve for any possible high stage of backwater be developed. The total length of this curve gives the limit of a short flume for decelerating flow. Any excess length classifies the flume as a long structure.

Detailed reasoning for this method of developing backwater and drop-down curves with numerous examples and the mathematical disclosure of the critical-depth limitations of the drop-down curve were given by Husted (*16*). It is to be noted that this method does not limit the shape of flume to that of a wide rectangle so that flow per unit width can be assumed. In the use of some tables of backwater functions, the limitation of unit flow is likely to be overlooked. Such tables are not adapted to use for many flume shapes. The method outlined above is not limited to any shape of section.

CRITICAL DEPTH

In a given flume of any shape, critical flow may occur in several ways:

(1) For a given flow, Q, there is but one critical depth and one critical velocity.

(2) For any given depth, there is a velocity V_c that will generate critical flow Q_c. In the foregoing (1) and (2) there is, of course, one point in common.

(3) For any given elevation of the energy gradient above the bottom of the flume, representing the energy content or total head, H, there is but one critical depth, d_c, one critical velocity, V_c, and one critical flow, Q_c; and this flow is a maximum as compared with that due to all other combinations of $d+h=H$. In theory critical flow is a highly desirable condition, the surface being smooth and glassy, turbulence at a minimum, and the jet quite transparent in clear water. In practice this condition is assured and attained at critical brinks; as just above the steep intake to a flume chute, just above the inlet transition where there is an excessive drop into a flume, or just above the outlet of a flume where there is a lower tail-water stage than $Z+h_r$. In practice this critical condition can be anticipated in design for very short flume sections, either raised or depressed, with respect to the canal sections at the ends. However, in practice, long flumes should not be designed for critical flow without due regard to the flat apex of the energy curve for any given Q. (See H, d curve in fig. 7.)

The energy curve and the momentum curve, near critical depth, indicate that the same quantity, Q, may flow at a rather wide range of depth with but little change in the total head $d+h=H$. In other words, if normal depth for a given value of n is set for critical depth, slight divergence of the actual value of n from the assumed value and slight retarding effects of curves will cause the surface of the water to assume a wide range of depth, both above and below the critical, with an agitated water surface.

The determination of critical depth in a flume problem may be made with various elements and conditions given:

(1) For a water prism of any shape, given the area, A, and the surface width, T, then

$$h_c=\frac{A}{2T} \tag{19}$$

from which

$$V_c=\sqrt{\frac{2gA}{2T}}=\sqrt{\frac{gA}{T}} \tag{20}$$

For a given width, T, of a rectangular channel and the flow, Q, the critical depth

$$d_c=\sqrt[3]{\frac{\left(\frac{Q}{T}\right)^2}{g}} \tag{21}$$

For a mean velocity of water, V, in a rectangular channel a depth, d_c, will make $V=V_c$, and

$$d_c=\frac{V^2}{g} \tag{22}$$

For a given total energy content $d+h=H$ above the bottom of a rectangular channel,

$$d_c=2/3H \tag{23}$$

Similarly, in a V-shaped channel:

$$d_c=4/5H \tag{24}$$

At any point in any channel, if $V<V_c$ the flow is at streaming stage; if $V>V_c$ the flow is at shooting stage.

For a given energy content, H, the flow, Q, is a maximum when $d_c+h_c=H$ and $V=V_c$

By continuity,

$$Q=AV_c=A\sqrt{\frac{gA}{T}} \tag{25}$$

In a rectangular channel,

$$Q_{\text{max.}}=3.09TH^{3/2} \tag{26}$$

Conversely H is a minimum when flow is critical, i.e.,

$$d_c+h_c=H_{\text{min}}. \tag{27}$$

THE HYDRAULIC JUMP

For a given value of total head or energy content, H, there are two combinations of $d+h=H$, representing two stages of flow. One of these is at ordinary streaming flow and the other at high-velocity shooting flow. When water has once passed critical velocity and entered the shooting stage, it can proceed at this stage if the slope is sufficient to maintain the high velocity developed. When, however, as often happens, this condition does not prevail the flow tends to pass through a rough, almost vertical uplift to the alternate stage having the same energy content. This action is called the "hydraulic jump." Many conditions may result in the development of the jump, but where it occurs in the flume itself (pls. 12, C and 15, B), the usual reason is the one given. The jump may be expected also at the outlets of all flumes flowing faster than the critical, such as chutes, except where a full overpour occurs.

However, in passing from the shooting to the streaming stage the true alternate stage is not reached, since there is always more of less loss of head in the jump. Much has been written concerning this loss and the jump has been extensively used to dissipate energy from high-velocity water. On the other hand, it has not been generally appreciated that there are also great possibilities in recovery of velocity head through the jump. A shallow, swiftly flowing water prism loses a large percentage of its energy in passing through the jump. On the other hand, a deep prism will lose but little energy and recover a large percentage of Δh. This condition holds where the energy curve and the momentum curve are similar in shape. (See fig. 7 for $Q=656$.)

The amount of the loss can not be determined by the energy curve, which itself is based on the law of conservation of energy, which does not recognize such losses. The law of conservation of momentum, however, is truly applicable. That is:

$$\frac{Qv}{g}+P=\frac{Qv'}{g}+P' \tag{28}$$

but since $v=Q/a$

$$\frac{Q^2}{ag}+P=\frac{Q^2}{a'g}+P' \tag{29}$$

where a and P apply to area and hydrostatic pressure before the jump (to flow at shooting stage) and a' and P' apply to the same elements after the jump (to flow at streaming stage). This equation gives the data for the so-called momentum curves, before and after the jump.

For any regular prism the hydrostatic pressure in the cross section in cubic units of water is given by: *(25a)*

$$P = \int A d d \tag{30}$$

Letting P represent the hydrostatic pressure in the cross section in cubic units of water.

For flumes of rectangular shape

$$P = \frac{bd^2}{2} \tag{31}$$

For flumes of triangular shape

$$P = \frac{1}{3} d^3 \tan \frac{1}{2} \theta \tag{32}$$

where θ is the angle of the flume notch. For the ordinary triangular flume, with sides at right angles to each other ($\theta = 90°$) $\tan \frac{1}{2}\theta$ becomes $\tan 45° = 1.0$; hence

For triangular shape with sides at 90°,

$$P = \frac{1}{3} d^3 \tag{33}$$

For circular shapes the formula becomes very complex. Tables for solution are found in reference (*31, p. 48a*).

In formula (29) if the two sides of the equation are platted for the two stages of flow, they will intersect at the point of common value, as indicated by the equation, and this intersection will indicate the longitudinal location of the jump in the channel. This point is rather clearly defined in a transition at the outlet of a flume, where the shape of the channel is changing rapidly, but there may be difficulty in locating the position of the jump in a flume section where the channel is uniform in shape and the cause of the jump may be nothing more than a local increase in the value of n, due to surface conditions or to excess retardation by excessive curvature or some other cause that can hardly be evaluated. Such a jump occurred in our example flume when the structure was new and normal flow was slightly faster than the critical, this jump being caused by gradual retardation of the water by four reverse curves until it jumped from shooting to streaming stage and then the velocity immediately commenced to accelerate down a tangent until at the outlet it had again become approximately critical (see pl. 6, A).

The algebraic expressions for the height of jump are somewhat complex for sectional shapes commonly used in flumes. However, the energy curve and the momentum curve can be quickly developed for any shape of section and the height of the jump can be determined graphically (fig. 7) and its longitudinal location in the channel can be determined by the intersection of the curves as described above. Hinds has described these curves and this method in great detail (*13*), and his suggestions have been abstracted by King (*18*).

NECESSARY FIELD DATA FOR THE COMPUTATION OF VARIOUS ELEMENTS ESSENTIAL TO THE STUDY OF FLOW CHARACTERISTICS IN FLUMES [8]

As previously suggested, long flumes and short flumes required different treatment. For the former, reaches from 1,000 to 4,000 feet long were selected and data were obtained for computation of retardation elements; influence of curvature; change of interior surface and

[8] In another bulletin (*24*) the writer gave the details of field data, equipment, and office procedure. Except where changes and improvement occurred, these details are not repeated here.

hence in friction factors, with the passage of time; similar changes during the summer season, largely due to algae or insect larvae. Sometimes loss of head at the inlet and/or at the outlet end could be obtained for long flumes; seldom were both obtained on the same long flume.

For short flumes it was customary to follow the water from the canal above through the inlet transition, then through the flume and outlet transition into the canal again. Exceptions occurred where definite breaks in the energy gradient evidenced by the hydraulic jump or by an overpour drop, spoiled an opportunity to record inlet or outlet losses.

For all tests for friction factors, the required elements were:

(1) The quantity of water, Q, comprising the steady flow necessary for sufficient time to make all the field measurements except the basic line of levels which can be run either before or after the hydraulic test.

(2) A profile of the water surface throughout the length considered.

(3) Measurements from which water-prism areas and perimeters can be computed or measured from large-scale graphs and the various values of the corresponding hydraulic radii computed.

(4) A profile of the bottom may be taken by levels or it may be developed from the measurements under item (3) above. The elevations of the bottom points are not used directly in the computations for retardation factors.

For practically all observations on the capacities of flumes, the quantity of flow, Q, has been determined with a current meter. In the work done by this Bureau, the meter station was usually located at the lower end of a long tangent and also was assumed to be the lower end of the reach tested. The standard measurement was by the vertical integration method with a Hoff meter[9] (pl. 10, A).

Verticals were taken at short intervals across the width of the flume. Soundings for depth were usually made with a slender current meter rod, measuring down to the water surface from the top front edge of a crosstie and then down to the bottom from the same point. The difference between these two measurements gave the depth of water without influence of velocity head causing the water to run up the rod. The soundings and point data for the meter measurement gave the cross section of the flume at the meter station, from which a could be computed and p for curved-section flumes measured with dividers on a large-scale plat of the section. For rectangular sections p could more easily be computed.

The determination of water-surface profile offered some difficulties. A glance at the views shown in the illustrations indicates the extent of surface roughness. The high velocities usually inherent in flumes and the rapid changes in velocities, and hence in surface elevations as water leaves a canal, is accelerated in entering a flume, flows through it and is then decelerated as it returns to the canal, are conducive to local fluctuations of the surface throughout a range often nearly 1 foot in elevation. The water surface in a long flume is usually smoother after normal flow has been approximated, but still is very rough for precision measurements necessary to a close determination of the slope.

As the flume usually has a foot walk, crossties, on which points are spotted and from which measurements down to the water surface are taken, are available at any selected station, the points being tied later into the scheme of levels.

[9] The Hoff current meter has a horizontal shaft with a propeller presenting a uniform face to the oncoming water. This meter is not affected to any noticeable extent by vertical movement in still water. Therefore any component of velocity due to a slow vertical movement in running water is probably negligible.

During the first tests a point gage was used. Later this was modified into a hook gage with adjustable slide, using the elbow of the hook rather than the point. The device finally adopted is shown in plate 10, A. In essentials the water surface is allowed to seek its own average level by entering the cylindrical well through four small piezometer orifices placed in pairs, 3 inches and 9 inches down the tube, thus obviating vacuum troubles as the flow is split by the rounded point. When taking an observation this tube, with a rounded, sealed end, is held pointing directly upstream and submerged for a few tenths of a foot. Pressure on the orifices causes water to rise in the well, rapidly at first, and gradually slowing down as pressures are equalized. In about 30 seconds the water is stilled in the well, the valve V is closed, and the gage is withdrawn with water trapped in the well. The hook gage in the well is then run to the water surface and the vernier reading recorded. This reading, taken in conjunction with the constant determined by the position of the gage on the meter rod and the elevation of the spotted point, gives an average damped-down elevation of the water surface at that section. At the upper and lower ends of a reach under test several of these readings were taken and the average was assumed to be the elevation of the water surface at that end. The sum of the velocity head for the velocity and the surface elevation at any given section gives a corresponding point on the energy line, E. The fall of the energy line, determined from the several points thus obtained, is the slope element desired.

The depth gage can be operated down from an elevation spot or up from the bottom to the water surface, using a pointed section on the current meter rod upon which the gage is threaded through attached rings. When measuring up from the bottom in trapezoidal canal sections above and below the flume, a level rod is clamped alongside the gage and is read by the instrument man while the water is entering the stilling well of the gage (pls. 10, A and 6, B).

The cross-section measurements were made in various ways, by rod and level for elevations, by steel tape and slender graduated metal rods for linear dimensions. Some of the large metal flumes were sectioned on the outside, by hanging a plumb bob over the edge and measuring offsets to the flume shell in both horizontal and vertical directions. Two plumbed vertical strings and a horizontal string formed a perfect U around the bottom of the flume.

In addition to the measurements outlined above, the notes were completed with such descriptive matter as pertained to the capacity of the flume, comments as to paint, algae, moss, detritus, curvature, flume surface. wind, and so on.

SCOPE OF EXPERIMENTS

Tests were made on flumes in Arizona, California, Colorado, Idaho, Nevada, Montana, Oregon, Washington, Wyoming, and Utah, and Alberta, Canada.

The range of materials included concrete—poured, precast, and gunite; metal—iron, steel, and alloys; and wood—plank and staves.

Sizes ranged from little structures but 1 foot wide to Brooks Aqueduct in Canada (no. 23 et seq.), 20 feet wide and 9 feet deep, and Tiger Creek conduit in California, some 20 miles long, 14 feet wide, and 6 feet deep. Alignment ranged from straight sections more than a mile long (no. 104) to reaches on Tiger Creek conduit with 413° of curvature in 1,000 feet of flume and with a minimum radius of but five times the width of the flume. The complete length of short flumes (from 50 feet up) was used in the tests. From very long flumes typical reaches of 1,000 or more feet were selected.

Velocities ranged from streaming flow as low as 1.08 feet per second through critical flow condition of some 11 feet per second up to shooting flow nearing 30 feet per second.

Interior conditions covered surfaces new and old, clean and algae-coated, painted and unpainted, smooth and rough.

The majority of the tests were made by field parties headed by the writer, or other engineers in this Division or its predecessors. Many of the data were abstracted from the files of the United States Bureau of Reclamation.

A few tests abstracted from engineering literature were computed on the basis of uniform flow, without sufficient field data to determine whether or not the areas of water prism at the ends of the reach were alike. Uniform flow is shown in table 3 to be approximated only, even in long reaches. These tests are given a lower rating in column 5, table 2, as complete measurements would usually have shown slightly different values of the friction factors. (See p. 25.)

OFFICE COMPUTATIONS AND EQUIPMENT

In solving any of the flow formulas for open channels to find the value of the retardation factor n, or its equivalent, it is necessary to compute:

(1) The mean velocity of the water, V, throughout the length of reach tested.

(2) The slope of the energy gradient, S, throughout the reach.

(3) The value of the mean hydraulic radius, R, throughout the reach.

For flows that do not involve the entrainment of air, the value of V was determined from the continuity equation $V=Q/A$ where Q was computed from the current meter notes and A was the weighted mean value of the local areas, $a_1, a_a, a_b, a_c \ldots a_2$. For tapered flow the mean value of local v_1, etc., is better if a great number of sections are measured.

The slope of the energy gradient is taken as the average rate of fall from the elevation of a point on the E line at the upper end of the reach, E_1, to that of a corresponding point at the lower end of the reach, E_2. Expressed in symbols,

$$h_f = (k_1+d_1+h_1) - (k_2+d_2+h_2) = (Z_1+h_1) - (Z_2+h_2) = E_1 - E_2 \quad (15) \text{ (p. 15)}$$

The mean rate of loss, $S=\frac{h_f}{L}$ (See table 2, column 15.) (14)[10] (p.15)

[10] Most works on hydraulics refer to the slope of the water surface as the element that induces flow in the direction of slope. This is true only where the flow is uniform and the kinetic energy, evidenced by the velocity head, is the same at the two ends of the reach. Of the hundreds of tests that have been made by the writer, on open channel flow, he does not recall one case where uniform flow existed throughout the reach. There appears to be a tapered flow in every case. As a rule this is not of great moment and quite often is negligible for experiments in ordinary earth channels in which velocities in excess of 3 feet per second are rare. When the computations for flumes were started, however, it immediately became evident that the changes in areas at the ends of the reach were sufficient to cause material net changes in the high velocities encountered, and consequently in velocity heads. In using Bernoulli's theorem $k_1+d_1+h_1=k_2+d_2+h_2+h_f$, hydraulic engineers generally have assumed that h_1 and h_2 were equal and therefore could be omitted from consideration. Many of the tests secured from other agencies and listed in tables 2 and 3 were originally computed for the slope of the water surface and, wherever possible have been corrected by adding the velocity heads to the water surface at the ends of the reach, thus developing the energy line, the fall of which determines the friction loss.

It is fully appreciated that this method assumes complete recovery of any velocity head released by the fact that areas toward the lower end were larger and the velocities less than at the upper end. This assumption does not appear erroneous as the taper of flow is usually quite gentle and forms a long, slim wedge in a channel of the same shape throughout, the ideal transition. Likewise it is assumed that all excess fall over that necessary to overcome friction has been converted into additional velocity, in cases where the area is less at the lower end of the reach and hence the velocity is greater.

The value of the mean hydraulic radius, R, is taken as the weighted mean value of r_1, r_a, r_b, r_c, . . . r_2. Each particular value of r is obtained by dividing the corresponding local area, a, by the wetted perimeter p. Except for uniform flow, this does not yield the same value as dividing the mean area, A, by the mean perimeter, P.

With V, S, and R, computed from field measurements, the value of Kutter's n was estimated from a diagram devised by the writer for the solution of the Kutter formula (fig. 3). It was then computed on an electric calculating machine and excess numerals discarded. The machine computations were repeated if they did not closely approximate the value of n as taken from the diagram.

The weight to be attached to any given test is largely proportional to the length of reach available for experiment. Short flumes and short reaches in long flumes yield final results that are not in conformity with those secured for similar conditions in long reaches. Inordinately low or high values of n may be found in short reaches.

Chute flumes (p. 89) are listed at the ends of the tables and were computed three ways so far as the field data warranted: (1) By using the velocity and the water-plus-air section as actually measured. This is the condition holding in the field. Where the velocity was not measured directly—usually by color—computations could not be made this way. (2) By using the measured section and a velocity as computed by assuming the $V=Q/A$ which does not represent true conditions for chute flumes. (3) By using the velocity as measured directly and computing a water section as though it included no air. Cone and associates (7) computed their results only in the second way; Steward[11] in the second and third ways. The quantity of flow, Q, was always measured either before or after travel down the chute.

Computations for the various problems were made with 20-inch slide rule.

ELEMENTS OF EXPERIMENTS FOR DETERMINATION OF COEFFICIENT OF FLOW IN CHEZY'S FORMULA AND RETARDATION FACTORS IN KUTTER'S AND MANNING'S FORMULAS

Tables 2 and 3 give the elements of all known acceptable observations on flumes of concrete, metal, or wood, arranged alphabetically in that order. The various series are generally placed in ascending order of values of Kutter's n. Need for close association or various tests on the same reach or same structure has sometimes suggested some other order as preferable. Table 3 is supplementary to and in the same order as table 2, items being identified by reference numbers in column 1.

[11] STEWARD, W. G. THE DETERMINATION OF N IN KUTTER'S FORMULA FOR VARIOUS CANALS, FLUMES, AND CHUTES ON THE BOISE PROJECT AND VICINITY. U.S. Department of the Interior, Reclamation Service. Report Boise Conference of Operating Engineers for Irrigation Canal Systems, located in Idaho, Oregon, and Washington. 1913. [Multigraphed.]

EXPLANATORY NOTES ON TABLE 2

Items in table 2 are listed according to types (p. 8) as follows:

Long flumes, then short flumes, usually at streaming velocities.

Flume chutes at shooting velocities are not numerous enough to separate by materials of construction. However, those that might class as long chutes are given first and in the order concrete, metal, and wood. Flows in chutes are computed in three ways and so listed: (1) By using the measured area of the wet prism (water plus air) and the corresponding perimeter and resulting hydraulic radius, but using a velocity, V, as measured directly by color or otherwise. This velocity is always higher than as computed below since there is always more or less entrainment of air. (2) By using measured sections but assuming that velocity, $V=Q/A$. (3) By using the measured velocity with computed sectional elements for a net water prism, excluding the swell due to entrained air.

Column 1 gives reference numbers to identify items in table 3 and in the text matter. The letter "a" following the number shows the data were obtained from some source other than the work of this Bureau. Column 2 gives the initials of experimenter and his series number, where identified in original sources.

Column 5 gives tests that were accepted, ratings A, B, or C. Where the conditions and equipment warranted field measurements that could be considered of the best, the work was given an A rating. Usually a B rating was for intermediate conditions or field measurements that did not yield data for the slope of the energy gradient. A rating of C was assigned where the data were just acceptable, especially for a short reach where the difference between surface slope and energy slope might be very appreciable and yield entirely different values of n, if computations were based on the former alone. In flume tests, the high velocities and rough-water surface make a close determination of the slope factor impossible without relatively precision equipment. A long reach and many surface measurements mitigate this difficulty, but on short flumes absurd values of Kutter's n are sometimes found.

Column 7 gives the shape of the flume. The principal joints of distinctive type are shown in figure 1. The shapes of sheet-metal flumes nearly always approximate that of the hydrostatic catenary. Exceptions occur for corrugated flumes. These are stiff enough so there is little sag in the center.

Column 8 shows the width of rectangular shapes and diameter of circular or shapes that approximate the circular. Columns 14 and 15 are developed in detail as columns 15 and 16 of table 3.

The other columns are believed to be self-explanatory when considered in connection with the notation and nomenclature beginning on page 5.

Further information regarding many of the flumes listed is contained in literature citations on pages 94 to 98.

EXPLANATORY NOTES ON TABLE 3

Column 1 corresponds with the same item in table 2. Column 2 shows the flow quantity, Q', used in design. When compared with column 3 this gives an idea of the relative "fullness" of the flume. Column 3 gives the flow measured at the time of the test observations. Columns 4 to 8, inclusive, and 10 to 14, inclusive, give the elements in order of development that finally result in the location of the energy gradient at the upper and lower ends of the reach respectively. It is to be noted that the corresponding elements are seldom even closely alike. The difference on columns 6 and 12 indicate the velocity-head change that must be applied to the slope of the water surface before correct values of the effective slope can be obtained. Column 16 is the resulting development of table 3 and is the slope element for final computations as listed in column 15 of table 2. Column 17, where known, shows the degree of conformity between the designed, constructed slope of the flume bed and the actual energy slope that existed at time of test. Note that one may be 2 or 3 times as great as the other.

Where items are lacking in columns showing the elements at the two ends of the reach, then these items could not be developed from the data available in these studies or had not been taken in the field. This condition indicates that the surface slope, or in some cases the bed slope was used in computations. These tests were always given a lower rating than would have been the case if the effect in change of velocity could have been developed.

TABLE 2.—*Elements of experiments on flumes for the determination of the coefficient of flow in Chezy's formula and retardation factors in Kutter's and Manning's formulas* [1]

LONG CONCRETE FLUMES

Reference number	Experimenter and series	Year of test	Age, approximate	Test rating	Name, location, and condition of flume or canal	Shape [2]	Width or diameter	Length of reach L	Mean elements, water prism: Flow Q	Area A	Velocity V	Hydraulic radius R	Retardation loss h_f	Energy slope $S=\frac{h_f}{L}$	Coefficients: Chezy C	Kutter n	Manning n'	Temperatures: Air °C.	Water °C.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
			Yrs.				*Feet*	*Feet*	*Sec.-ft.*	*Sq. ft.*	*Ft. per sec.*	*Feet*	*Feet*	*Ft. per ft.*					
1a	USBR-D	1922	0.2	A	C-Canal flume Klamath Falls, Oreg	Rect.	11.0	3,953.0	307.2	48.73	6.30	2.45	2.69	0.000680	154.4	0.0113	0.0111	-----	-----
2a	do	1922	.3	A	Same, 3 weeks later on July 17	Rect.	11.0	4,168.0	133.2	30.03	4.44	1.82	2.30	.000552	140.1	.0120	.0117	-----	-----
3a	do	1922	.4	A	Same, Aug. 29	Rect.	11.0	4,168.0	40.8	14.73	2.77	1.08	2.42	.000582	110.5	.0148	.0136	-----	-----
4a	do	1923	1.1	A	Same, June 6	Rect.	11.0	3,988.0	210.2	40.32	5.21	2.20	2.43	.000609	142.4	.0121	.0119	-----	-----
5a	do	1923	1.2	A	Same, July 11	Rect.	11.0	3,276.0	99.6	23.73	4.20	1.55	2.11	.000644	132.9	.0126	.0120	-----	-----
6a	do	1923	1.2	A	Same, July 17	Rect.	11.0	3,278.0	83.1	21.09	3.94	1.42	2.07	.000631	131.6	.0123	.0120	-----	-----
7a	do	1923	1.3	A	Same, Aug. 15	Rect.	11.0	3,278.0	178.5	34.68	5.15	2.00	2.28	.000695	138.1	.0123	.0121	-----	-----
8a	do	1923	1.4	A	Same, Aug. 22	Rect.	11.0	3,280.0	124.7	28.82	4.33	1.78	2.05	.000625	129.8	.0128	.0126	-----	-----
9a	do	1923	1.4	A	Same, Aug. 29	Rect.	11.0	3,280.0	154.2	33.73	4.57	1.97	1.92	.000584	134.7	.0126	.0124	-----	-----
10a	EGH	1909	0	B	Tieton flume, Wash	Circ.	8.3	(3)	274	-----	-----	-----	-----	.00165	-----	.012	-----	-----	-----
11a	do	1909	0	B	do	Circ.	8.3	(4)	160	-----	-----	-----	-----	.00165	-----	.012	-----	-----	-----
12a	USBR	1922	-----	---	Henley, Klamath Falls, Oreg	Rect.	11.0	2,174.0	40.8	13.61	3.00	1.01	1.472	.000677	114.9	.0131	.0129	-----	-----
13	S-308	1923	3	A	Main, King Hill project, Idaho	Rect.	8.58	1,061.0	209.1	27.06	7.74	1.816	1.919	.001808	135.1	.0124	.0122	28	20
14	S-309	1923	3	A	do	Rect.	8.58	1,206.0	209.1	27.48	7.61	1.857	2.364	.001899	128.1	.0133	.0129	28	20
15	S-352	1928	1	A	Cross-over flume, Pacific Gas & Electric Co.	Rect.	4.0	1,950.0	33.3	6.85	4.86	.97	3.87	.001985	111.1	.0135	.0132	-----	15
16	S-300a	1919	2	A	High line, Lindsay-Strathmore, Calif	Rect.	5.75	1,172.4	42.88	11.47	3.74	1.225	1.120	.000955	109.4	.0142	.0141	43	22
17	S-300b	1919	2	A	Same, next day	Rect.	5.75	1,172.4	44.4	11.79	3.77	1.244	1.130	.000964	108.9	.0143	.0143	42	22
18	S-359a	1927	2	A	Orchard Mesa district, Colo	Rect.	17.0	1,700.0	372.0	69.13	5.38	2.77	1.35	.0007941	114.7	.0154	.0154	-----	-----
19	S-359b	1927	2	B	Same, sides cement washed, bottom tarred.	Rect.	17.0	800.0	372.0	71.35	5.21	2.83	.51	.0006375	122.7	.0145	.0144	-----	-----
20	S-359c	1927	2	B	Same, no moss, muddy slime at surface	Rect.	17.0	1,100.0	372.0	75.22	4.95	2.93	.69	.0006273	115.5	.0154	.0154	-----	-----
21	H-29	1913	1	A	Hamilton Flour Mill flume, Mont	Rect.	7.0	3,000.0	107.6	27.94	3.85	1.94	1.83	.000610	111.9	.0146	.0152	-----	-----
22	S-335	1923	9	A	Yakima Valley Canal, Yakima, Wash	Rect.	5.6	1,100.0	66.58	14.18	4.70	1.416	1.635	.001486	102.5	.0154	.0154	34	19.5

23	S-350a	1926	12	A	Brooks Aqueduct, Canadian Pacific Railway project, Alberta, Canada.	H.C.	20.3	1,361.0	217.0	65.83	3.30	3.06	.350	.000257	117.6	.0152	.0152	23	19
24	S-350b	1926	12	A	Same, day before	H.C.	20.3	1,361.0	497.2	105.27	4.72	4.00	.538	.000395	118.6	.0156	.0158	23	19
25a	A.G.-3A	1931	17	A	Same, 5 years later	H.C.	20.3	2,660.0	472.0	106.5	4.43	4.00	1.01	.000380	113.7	.0164	.0165	----	----
26a	A.G.-2A	1931	17	A	Same reach, includes S-350	H.C.	20.3	2,660.0	629.0	124.7	5.04	4.34	1.08	.000406	120.1	.0157	.0158	----	----
27a	A.G.-3B	1931	17	A	Just below reach A	H.C.	20.3	3,000.0	472.0	105.3	4.48	3.98	1.20	.000400	112.3	.0166	.0167	----	----
28a	A.G.-2B	1931	17	A	do	H.C.	20.3	3,000.0	629.0	121.1	5.20	4.29	1.33	.000443	119.1	.0158	.0159	----	----
29a	A.G.-1B	1931	17	A	do	H.C.	20.3	3,000.0	699.0	125.3	5.58	4.37	1.36	.000453	125.4	.0150	.0152	----	----
30a	A.G.-3C	1931	17	A	Just below reach B	H.C.	20.3	1,702.0	472.0	102.8	4.59	3.93	.67	.000394	116.7	.0159	.0160	----	----
31a	A.G.-2C	1931	17	A	do	H.C.	20.3	1,702.0	629.0	115.6	5.44	4.18	.71	.000417	130.3	.0144	.0145	----	----
32a	A.G.-1C	1931	17	A	do	H.C.	20.3	1,702.0	699.0	119.8	5.84	4.26	.76	.000446	133.9	.0140	.0141	----	----
33a	A.G.-3D	1931	17	A	Just below reach C and just above inverted siphon at railroad crossing.	H.C.	20.3	1,398.0	472.0	101.1	4.67	3.91	.58	.000415	115.9	.0160	.0161	----	----
34a	A.G.-2D	1931	17	A		H.C.	20.3	1,398.0	629.0	111.0	5.67	4.11	.73	.000522	122.4	.0153	.0154	----	----
35a	A.G.-1D	1931	17	A		H.C.	20.3	1,398.0	699.0	114.1	6.13	4.19	.76	.000544	128.3	.0146	.0147	----	----
36a	A.G.-3E	1931	17	A	Just below siphon	H.C.	20.3	1,120.0	472.0	94.4	5.00	3.76	.62	.000554	109.6	.0169	.0169	----	----
37a	A.G.-2E	1931	17	A	Same, note drop-down effect in C, D, E	H.C.	20.3	1,120.0	629.0	97.0	6.48	3.79	.85	.000759	120.8	.0153	.0154	----	----
38a	A.G.-1E	1931	17	A	do		20.3	1,120.0	699.0	94.2	7.42	3.71	.87	.000777	138.2	.0133	.0134	----	----
39	S-370-a	1931	.2	A	Tiger Creek Conduit stations, 927+50–916+50.	Rect.	14.3	1,100.0	537.0	78.6	6.84	3.26	1.114	.001013	119.0	.0152	.0152	[5] 85	----
40	S-370-b	1931	.2	A	Same, Pacific Gas & Electric Co., 916+50–899+00.	Rect.	14.3	1,632.8	537.0	73.7	7.29	3.15	1.764	.001081	124.9	.0144	.0144	[5] 267	----
41	S-371-a_1	1931	.2	A	Same, Mokelumne River, 605+50–594+25	Rect.	14.3	1,125.0	515.0	75.7	6.80	3.19	.966	.000859	129.9	.0139	.0139	[5] 375	----
42	S-371-b_2	1931	.2	A	Same, California, 594+25–583+25	Rect.	14.3	1,100.0	515.0	77.2	6.67	3.23	.780	.000709	139.4	.0129	.0130	[5] 226	----
43	S-371-c_2	1931	.2	A	Same, August, 583+25–566+50	Rect.	14.3	1,671.4	515.0	77.2	6.68	3.23	1.285	.000769	134.0	.0135	.0135	[5] 512	----
44	S-371-b_1	1931	.2	A	Same, August, 594+00–583+25	Rect.	14.3	1,075.0	540.0	79.7	6.78	3.28	.730	.000679	143.7	.0126	.0126	[5] 226	----
45	S-371-c_1	1931	.2	A	Same, August, 583+25–566+50	Rect.	14.3	1,671.4	540.0	79.6	6.78	3.28	1.290	.000772	134.7	.0134	.0134	[5] 512	----
																			°F.
46	S-371-b+c-5	1932	.9	B	Same, April, no algae, 596+25–568+00	Rect.	14.3	2,890.5	102.5	22.82	4.49	1.41	2.309	.000799	134.0	.0121	.0118	[5] 737	38
47	S-371-b+c-4	1932	.9	B	Same, day before, no algae, 596+25–568+00	Rect.	14.3	2,890.5	220.0	40.03	5.50	2.15	2.228	.000771	135.1	.0127	.0125	[5] 737	38
48	S-371-b+c-3	1932	.9	B	do	Rect.	14.3	2,890.5	322.0	52.07	6.18	2.56	2.176	.000753	140.9	.0125	.0123	[5] 737	38
49	S-372-a_1	1931	.2	A	Same, lower end flume, 66+00–55+50	Rect.	14.3	1,050.0	516.0	69.7	7.41	3.05	.750	.000714	158.8	.0113	.0112	[5] 139	----
50	S-372-a_2	1931	.5	B	Same, November, 63+00–55+50	Rect.	14.3	750.0	289.4	45.30	6.39	2.33	.495	.000660	162.8	.0107	.0105	[5] 87	----
51	S-372-b_1	1931	.2	A	Same, November, 55+50–44+00	Rect.	14.3	1,150.0	516.0	69.6	7.42	3.05	1.080	.000939	138.5	.0130	.0130	[5] 461	----
52	S-372-b_2	1931	.5	B	Same, no algae, 55+50–44+50	Rect.	14.3	1,100.0	289.4	44.35	6.53	2.30	.980	.000891	144.4	.0121	.0118	[5] 455	----
53	S-372-c_1	1931	.2	A	Same, no algae, 44+00–33+00	Rect.	14.3	1,100.0	516.0	68.5	7.54	3.02	.895	.000809	152.6	.0118	.0118	[5] 250	----
54	S-372-c_2	1931	.5	B	Same, no algae, 44+50–33+50	Rect.	14.3	1,099.2	289.4	43.48	6.66	2.27	.960	.000874	149.6	.0116	.0114	[5] 236	----

[1] Experiments by men in this bureau identified as follows: S to the writer, Fred C. Scobey, H to S. T. Harding; C-T-J to V. M. Cone, R. E. Trimble, and P. S. Jones; B to D. H. Bark; WBG to W. B. Gregory. Most of the experiments by other agencies were made by various engineers of the U. S. Bureau of Reclamation, identified USBR. Where these data could be further identified, the initials follow. D to A. L. Darr; K-H to H. W. King and the late E. G. Hopson; R to Paul Rothi; F to L. J. Foster; S to W. G. Steward and associates. Others from whom tests were received were: JDG to J. D. Galloway, consulting engineer, and associates; S. F. to S. Fortier, then with U.S. Geological Survey; WBD to W. B. Dougall, associated with Dr. Fortier. K to A. W. Kidder and G. to E. A. Garland, engineers of the Pacific Gas & Electric Co., Calif.; A. G. to Augustus Griffin, chief engineer, Department of Natural Resources, Canadian Pacific Railway Co.; Densmore to J. P. Densmore, engineer of the Southern California Edison Co. of California; J. E. to J. Eppler of Switzerland; EGH to E. G. Hopson.

[2] Rect.=rectangular; H.C.=hydrostatic catenary; Circ.=semicircular; and Trap.=trapezoidal. For metal flumes, the size number, equal to the lengths in inches of the sheets transverse to the flume is followed by the initial of the name type as known in irrigation practice: L for Lennon, H for Hess, Hn for Hinman, K for Klaur, T for Thompson, G for Gage, M for Maginnis.

[3] 2.5 miles.

[4] 4 miles.

[5] Degrees of curvature. For Tiger Creek Conduit the extent of the curvature is a factor in the coefficients as computed. The total curvature within each reach, L, is given above.

TABLE 2.—*Elements of experiments on flumes for the determination of the coefficient of flow in Chezy's formula and retardation factors in Kutter's and Manning's formulas*—Continued

SHORT CONCRETE FLUMES

Reference number	Experimenter and series	Year of test	Age, approximate	Test rating	Name, location, and condition of flume or canal	Shape	Width or diameter	Length of reach	Mean elements, water prism						Coefficients			Temper-tures	
									Flow	Area	Velocity	Hydraulic radius	Retardation loss	Energy slope	Chezy	Kutter	Manning	Air	Water
								L	Q	A	V	R	h_f	$S=\frac{h_f}{L}$	C	n	n'	°*C.*	°*C.*
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
			Yrs.				*Feet*	*Feet*	*Sec.-ft.*	*Sq. ft.*	*Ft. per sec.*	*Feet*	*Feet*	*Ft. per ft.*					
70	S-311	1923	3	B	McEachren flume, King Hill project, Idaho.	Rect.	8.83	752.0	193.6	26.28	7.37	1.802	1.416	0.001883	126.6	0.0132	0.0130	24.6	16.8
71	S-362	1929	14	A	Anderson Creek flume Anderson-Cottonwood Irrigation District.	Rect.	8.0	1,124.3	129.1	25.13	5.14	1.802	1.035	.000921	126.2	.0132	.0130	38	21
72	S-369	1928	1	B	Tahoe flume Drum canal, Sierra Nevada	Rect.	10.0	1,450.0	494.5	52.18	9.477	2.70	2.624	.001876	133.1	.0133	.0129	----	----
73	S-369a	1928	1	C	Same, Pacific Gas & Electric Co	Rect.	10.0	626.0	494.5	53.32	9.27	2.73	1.015	.001660	137.7	.0129	.0128	----	----
74	S-369b	1928	1	C	do	Rect.	10.0	824.0	494.5	51.30	9.64	2.68	1.609	.002039	130.4	.0135	.0130	----	----
75	J.E	1902	----	C	Flume, Simplon Tunnel, Alps, Europe	Rect.	3.30	164.2	39.6	4.95	8.00	.782	1.202	.007320	105.7	.0137	.0135	----	----
76a	K-H.c	1909	2	B	Circular trough, Umatilla project, Oreg.	Circ.	9.8	640.0	205.0	28.50	7.19	2.13	1.04	.001625	122.3	.0140	.0138	----	----
77a	K-H.d+e	1909	2	C	Same, sharp reverse curves	Circ.	9.8	340.0	205.0	29.35	6.98	2.16	.73	.002147	104.4	.0163	.0166	----	----
78a	K-H.f	1909	2	B	Same, relatively straight	Circ.	9.8	1,075.0	205.0	28.45	7.21	2.12	1.89	.001758	118.4	.0144	.0142	----	----
79a	K-H.c+d+e+f	1909	2	B	Same, combines above reaches	Circ.	9.8	2,055.0	205.0	28.98	7.07	2.14	3.66	.001781	114.6	.0149	.0147	----	----
80	S-306	1923	6	A	Ridenbaugh Canal, Boise, Idaho	Rect.	7.0	420.5	30.1	9.78	3.083	1.042	.386	.000918	99.7	.0150	.0150	25	21
81	S-342	1924	8	B	Dry Creek flume Modesto Irrigation District, Calif.	Rect.	18.0	579.9	247.0	96.86	2.55	3.37	.186	.0003208	77.6	.0228	.0235	----	----

LONG METAL FLUMES—FLUSH INTERIORS

98	S-302	1923		A	Des Chutes municipal district-Oreg	204L	10.82	1,800.0	80.75	12.91	6.26	1.359	1.72	0.000956	173.8	0.0094	0.0090	18	14
99	S-302	1926		A	Same, 3 years later	204L	10.82	1,806.0	70.72	14.09	5.02	1.433	1.839	.001018	131.4	.0123	.0120	18	14
100	C-T-J	1913	New	B	Garland flume Trinchera Irrigation District, Colo.	168L	8.9	2,850.0	19.6	4.23	4.63	2.669	7.05	.002474	117.4	.0122	.0122		
101	S-331A	1921	1	A	Selah-Moxee Main flume, Yakima, Wash.	132L	±6.80	1,500.0	55.41	14.39	3.85	1.501	.949	.000633	125.1	.0129	.0127	26	20
102	S-331B	1923	3	A	Same (just cleaned)	132L	±6.80	995.7	65.9	14.58	4.52	1.505	.623	.000625	147.4	.0111	.0108	30	21
103	S-303	1923	9	A	Main flume no. 1, Tumalo Irrigation District, Oreg.	180L	9.25	1,800.0	159.9	26.92	5.941	2.039	1.782	.000990	132.2	.0127	.0127	30	13
104	S-364	1929	1	A	Agua Fria flume near Phoenix, Ariz	252L	13.4	2,160.0	314.0	56.44	5.56	2.967	1.323	.000613	130.4	.0137	.0137	24	21
105	S-361A	1929	2	A	Bowman-Spaulding Conduit, Calif	180+L	9.25	1,113.0	212.6	35.33	6.02	2.40	1.443	.00130	107.8	.0160	.0160	23	8
106	S-361B	1929	2	B	Same, flume size 180 plus raise	180+L	9.25	1,150.0	212.6	36.43	5.84	2.44	1.377	.00120	107.9	.0161	.0160	23	8
107	S-361C	1929	2	B		180+L	9.25	1,700.0	212.6	35.02	6.07	2.40	2.019	.00119	113.6	.0153	.0151	23	8
108	S-361D	1929	2	B	Same [6]	180+L	9.25	3,370.0	212.6	34.88	6.09	2.44	4.30	.00128	109.0	.0159	.0158	23	8
109	S-361E	1929	2	B		180+L	9.25	3,651.0	212.6	31.49	6.75	2.28	4.546	.00125	126.4	.0136	.0135	23	8

LONG METAL FLUMES—PROJECTING BANDS

110	S-337A	1923	3	B	Flume no. 1 east lateral Talent Irrigation District, Oreg.	108G	5.75	970.0	19.0	5.01	3.78	0.856	1.425	0.00147	106.7	0.0137	0.0136		
111	S-337B	1923	3	B	do	108G	5.75	229.0	19.0	5.00	3.79	.854	.323	.00141	109.2	.0134	.0133		
112	S-354	1926	4	A	No. 1. flume Tule-Baxter Irrigation District, Calif.	168G	8.9	1,003.0	121.8	19.08	6.38	1.702	1.856	.001850	113.7	.0145	.0143		21
113	S-325A	1923	13	A	Sleeping Child Creek flume, Hamilton, Mont.	168M	8.8	1,984.3	74.9	20.84	3.59	1.801	1.614	.000814	93.8	.0174	.0175		
114	S-325B	1923	13	B	Same, Hedge Canal, Mont	168M	8.8	565.0	74.9	22.48	3.33	1.867	.451	.000798	86.3	.0190	.0191		
115	S-325C	1923	13	A	do	168M	8.8	1,419.3	74.9	20.19	3.71	1.776	1.166	.000822	97.4	.0168	.0168		

LONG CORRUGATED METAL FLUMES

117	C-T-J	1913	3	B	Stewart, near Paonia, Colo	132	7.0	1,735.0	14.7	7.67	1.918	1.043	1.531	0.000882	63.2	0.0224	0.0237		

[6] Reaches follow order of letters after S-361. Algae at upper end, none here.

TABLE 2.—*Elements of experiments on flumes for the determination of the coefficient of flow in Chezy's formula and retardation factors in Kutter's and Manning's formulas*—Continued

SHORT METAL FLUMES—FLUSH INTERIORS

Reference number	Experimenter and series	Year of test	Age, approximate	Test rating	Name, location, and condition of flume or canal	Shape	Width or diameter	Length of reach	Mean elements, water prism						Coefficients			Temperatures	
									Flow	Area	Velocity	Hydraulic radius	Retardation loss	Energy slope	Chezy	Kutter	Manning	Air	Water
								L	Q	A	V	R	h_f	$S=\frac{h_f}{L}$	C	n	n'	°C.	°C.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
			Yrs.				*Feet*	*Feet*	*Sec.-ft.*	*Sq. ft.*	*Ft. per sec.*	*Feet*	*Feet*	*Ft. per ft.*					
120a	USBR-R	1915	3	B	Middle Creek low line no. 12, Nebr	108Hn	5.75	525.0	39.0	6.80	5.74	1.02	0.619	0.001179	165.5	0.0095	0.0090	----	----
121a	USBR-R	1915	3	B	do	108Hn	5.75	525.0	14.5	3.38	4.29	.68	.618	.001177	152.2	.0097	.0091	----	----
122a	USBR-R	1915	3	C	West Water low line no. 8 F., Nebr	132Hn	7.0	400.0	56.0	8.20	6.83	1.11	.507	.001268	182.3	.0089	.0083	----	----
123a									45.1	8.83	5.11	1.15	.393	.000982	151.8	.0105	.0100	----	----
124	S-103a	1917	2	B	Spider Coulee flume, Milk River project, Mont.	228H	12.0	262.7	194.1	27.69	6.91	2.050	.214	.000815	169.0	.0101	.0099	----	----
125	S-103b							524.0	226.2	31.08	7.27	2.171	.426	.000813	173.0	.0100	.0098	----	----
126	S-103c							524.0	237.0	32.42	7.31	2.220	.484	.000924	161.8	.0105	.0105	----	----
127	S-366	1930	2	C	Del Puerto flume-5S, West Stanislaus Irrigation District, Calif.	84L	4.5	55.6	11.02	4.38	2.52	.802	.023	.000414	138.3	.0107	.0104	----	----
128a	USBR-R	1915	3	B	Wild Horse low line no. 4 flume, Nebr	156Hn	8.33	600.0	78.0	12.08	6.46	1.35	.808	.001347	151.5	.0107	.0103	----	----
129a									95.0	13.58	7.00	1.44	.778	.001297	162.0	.0102	.0098	----	----
130a									64.0	10.70	5.98	1.26	.663	.001105	160.2	.0101	.0096	----	----
131a									67.7	11.35	5.96	1.30	.688	.001147	154.1	.0105	.0101	----	----
132a									44.3	9.28	4.77	1.16	.698	.001163	130.2	.0120	.0117	----	----
133	C-T-J	1913	3	B	Minnesota Canal, Colo	168L	8.9	200.0	7.04	1.17	6.01	.374	.959	.00480	142.0	.0088	.0086	----	----
134								60.0	7.04	1.17	6.01	.374	.574	.00957	100.6	.0125	.0125	----	----
135	C-T-J	1913	New	C	Garland, see no. 100 above	168L	8.9	450.0	19.6	3.62	5.41	.608	.885	.001966	156.5	.0093	.0087	----	----
136								650.0	19.6	3.66	5.34	.610	1.345	.00207	150.3	.0092	.0091	----	----
137								325.0	19.6	4.45	4.40	.691	.815	.00251	105.6	.0134	.0132	----	----
138a	USBR-R	1915	3	B	Red Willow low line no. 11 flume	120Hn	6.38	690.0	40.0	9.15	4.37	1.19	.614	.000890	134.2	.0117	.0114	----	----
139a									18.1	6.03	3.00	.94	.421	.000610	125.2	.0120	.0118	----	----
140	S-334B	1923	12	A	Nachez-Selah Irrigation District, Wash	156L	8.3	963.0	112.5	19.93	5.64	1.783	.845	.000878	142.7	.0117	.0115	33	24

141	S-367	1930	New	B	Escaladian flume, Merced Irrigation District, Calif.	240L	12.73	800.8	259.0	42.20	6.14	2.56	.534	.000666	148.7	.0118	.0120	29	15
142a	USBR-F2	1915	3	A	West Canal, Happy Canyon, Uncompahgre project, Colo.	192H	10.17	350.0	36.5	15.24	2.40	1.365	.08	.000230	135.6	.0117	.0116	-----	-----
143a	USBR-F3	1915	3	A	West Canal, Spring Creek, Uncompahgre project, Colo.	168H	8.92	300.0	20.8	10.18	2.04	1.170	.09	.0003	107.5	.0139	.0140	-----	-----
144a	USBR-F1	1915	5	B	King Lateral, Uncompahgre, Colo	120	6.35	197.0	62.5	7.77	8.04	1.069	-------	.003604	129.5	.0122	.0116	-----	-----
145a	USBR	1914	2	C	Happy Canyon flume, Uncompahgre project (R).			250.0	184	29.21	6.30	2.10	-------	.00094	141.8	.0121	.0119	-----	-----
146a	USBR	1914	2	C	Same (L) double-barreled flume			250.0	177	28.38	6.23	2.06	-------	.00111	130.3	.0131	.0129	-----	-----
147	S-316a	1923	11	B	Same (R)	204H	10.82	241.3	227.5	37.94	6.00	2.45	.247	.001024	119.7	.0145	.0144	23	15
148	S-316a	1923	11	B	Same (L)			241.3	238.0	37.85	6.29	2.448	.227	.000941	131.0	.0133	.0132	23	15
149	S-316A	1927	15	C	Same (R)			241.3	224.9	34.26	6.56	2.328	.197	.000816	150.5	.0116	.0114	-----	-----
150	S-316B	1927	15	C	Same (L)			241.3	223.1	34.42	6.48	2.329	.104	.000431	204.6	.0086	.0084	-----	-----
151	S-321	1923	13	C	Lockwood Irrigation District no. 1, Billings, Mont.	72K	3.82	184.0	14.7	4.68	3.13	.867	.138	.000750	122.9	.0121	.0118	31	24
152	S-322	1923	13	C	Lockwood Irrigation District no. 2, Billings, Mont.	72K	3.82	139.3	14.7	3.81	3.86	.768	.142	.001019	137.9	.0114	.0103	31	24
153	S-323	1923	13	C	Lockwood Irrigation District no. 3, Billings, Mont.	72K	3.82	302.3	19.1	5.57	3.43	.946	.653	.00216	75.9	.0188	.0192	31	24
154	S-324	1923	13	C	Flume no. 2, Upper Canal, Lockwood Irrigation District, Mont.	72K	3.77	92.0	20.4	6.70	3.04	1.029	.057	.000620	120.5	.0126	.0124	-----	-----
155	S-358	1926	New	A	E. C. Hauser flume, Calif	48L	2.55	110.6	8.36	1.32	6.32	.447	.861	.007785	107.1	.0123	.0121	31	20
156	S-377	1932	1	A	Kern River flume, Borel Canal, Calif	276L	14.64	1,160.7	603.0	65.48	9.21	3.17	1.43	.001232	146.8	.0123	.0122	-----	-----
157	S-376	1932	1	B	Rich Gulch flume, Borel Canal, Calif	360L	19.1	442.9	603.0	118.3	5.10	4.26	.172	.000388	125.4	.0150	.0151	-----	-----
158	S-312	1923	New	B	Murtaugh Canal, Milner, Idaho	144	7.65	227.4	91.0	14.25	6.39	1.494	.374	.001645	128.8	.0124	.0123	-----	-----
159a									105.0	17.77	5.91	1.603	.356	.001187	135.5	.0122	.0119	-----	-----
160a	USBR-R	1915	3	C	Hope Creek no. 1 low line flume, Nebr	204Hn	10.82	300.0	99.0	17.81	5.56	1.59	.362	.001207	126.8	.0129	.0127	-----	-----
161a									150.0	25.40	5.91	1.96	.31	.001047	130.3	.0130	.0128	-----	-----
162	S-360A	1927	11	B	Indian Wash flume, Grand Valley project, Colo., right barrel.	252H	13.5	85.0	216.1	34.55	6.25	2.261	.1054	.001235	118.3	.0145	.0144	32	21
163	S-360B				Same, left barrel				209.2	35.24	5.94	2.291	.073	.000859	133.9	.0129	.0127	32	21
164	S-319	1923	0.5	C	Jessup cut flume	168T	8.95	161.6	52.8	13.17	4.01	1.395	.110	.000681	130.2	.0126	.0121	23	24
165	S-363	1929	1	B	Risley flume, Volta, Calif	144L	7.62	375.0	108.8	18.18	5.98	1.695	.538	.001435	121.3	.0136	.0134	31	12
166	S-353A	1926	1	A	Erskine Creek flume, Borel Canal, Calif	348L	18.46	820.2	564.0	113.3	4.98	4.23	.265	.000323	134.7	.0139	.0140	28	15
167	S-353B	1932	7	A	Same, 6 years later	348L	18.4	820.2	602.0	121.8	4.94	4.367	.271	.000330	130.1	.0145	.0146	-----	-----
168	S-368	1930	2	A	LeGrand flume no. 1, Merced Irrigation District, Calif.	240L	11.3	1,150.0	295.0	48.32	6.10	2.745	1.028	.000894	123.0	.0144	.0143	29	15
169	S-365A	1930	18	B	Orland high line, Calif. (L)	192H	10.0	288.8	98.0	25.56	3.83	1.975	.151	.000523	119.3	.0141	.0140	22	18
170	S-365B	1930	18	B	Orland high line, Calif. (R)	192M	10.15	288.8	83.3	25.25	3.30	1.945	.259	.000896	79.0	.0208	.0210	22	18
171	S-348	1924	14	C	Winona flume, Canadian Pacific Railway project, Alberta, Canada.	132	7.0	169.9	14.7	3.55	4.14	.666	.424	.002496	101.5	.0137	.0140	19	16
172	S-336	1923	3	C	Talent Irrigation District, Oregon-Straight	108G	5.75	73.5	9.63	4.46	2.16	.798	.048	.000653	94.6	.0149	.0151	30	21
173	S-318	1923	11	B	Happy Canyon flume, near Montrose, Colo.	192H	10.1	321.3	102.5	33.13	3.10	2.297	.132	.000411	100.9	.0169	.0169	-----	-----

TABLE 2.—*Elements of experiments on flumes for the determination of the coefficient of flow in Chezy's formula and retardation factors in Kutter's and Manning's formulas* —Continued

SHORT METAL FLUMES—PROJECTING BANDS

Reference number	Experimenter and series	Year of test	Age, approximate	Test rating	Name, location, and condition of flume or canal	Shape	Width or diameter	Length of reach L	Mean elements, water prism: Flow Q	Area A	Velocity V	Hydraulic radius R	Retardation loss h_f	Energy slope $S=\frac{h_f}{L}$	Coefficients: Chezy C	Kutter n	Manning n'	Temperatures: Air °C.	Water °C.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
			Yrs.				*Feet*	*Feet*	*Sec.-ft.*	*Sq. ft.*	*Ft. per sec.*	*Feet*	*Feet*	*Ft. per ft.*					
176	S-308	1923	14	C	Mora flume, near Nampa, Idaho	180M	9.55	200.0	220.8	31.99	6.90	2.243	0.521	0.002605	90.3	0.01885	0.0188	14	13
177a	USBR-F4	1915	5	B	King lateral, Uncompahgre, Colo	108M	5.73	140.0	42.1	8.66	4.85	1.146	----	.002786	85.8	.0175	.0177	----	----
178	C-T-J	1913	3	C	Flume no. 3, King lateral, Uncompahgre, Colo., project.	108M	5.73	300.5	39.2	7.32	5.35	1.072	1.027	.003425	88.3	.0169	.0170	----	----
179	C-T-J	1912	3	C	do	120M	5.73	189.0	33.3	5.77	5.77	.916	.788	.00417	93.4	.0157	.0157	----	----
180	C-T-J	1913	3	C	Flume no. 10, King lateral, Uncompahgre, Colo., project.	96M	5.09	635.0	23.7	4.65	5.10	.841	2.732	.00430	84.8	.0168	.0170	----	----
181a	USBR-R	1915	3	B	Lateral 27 M, North Platte project, Nebr.	132M	7.0	590.0	38.5	9.85	3.91	1.24	1.104	.001871	81.2	.0187	.0190	----	----
182a								590.0	23.0	6.88	3.34	1.02	.959	.001625	82.3	.0178	.0181	----	----
183a								590.0	17.0	5.70	2.98	.92	.910	.001542	79.3	.0180	.0185	----	----
184a								490.0	75.8	16.76	4.53	1.62	.991	.002022	79.0	.0202	.0204	----	----

SHORT CORRUGATED METAL FLUMES

Reference number	Experimenter and series	Year of test	Age, approximate	Test rating	Name, location, and condition of flume or canal	Shape	Width or diameter	Length of reach L	Flow Q	Area A	Velocity V	Hydraulic radius R	Retardation loss h_f	Energy slope S	Chezy C	Kutter n	Manning n'	Air °C.	Water °C.
187	S-320A	1923	----	C	Carwile flume, Buffalo, Wyo	72	3.9	175.0	3.0	0.80	3.73	0.313	1.370	0.007829	75.3	0.0172	0.0163	----	----
188	C-T-J	1913	3	C	Fire Mountain Canal, Colo	144	7.61	249.6	16.9	8.84	1.91	1.11	.252	.00101	57.1	.0249	.0265	----	----
189	S-338	1924	----	C	Flume no. 1, Gage Canal, Riverside, Calif.	----	7.25	73.6	44.0	11.73	3.75	1.334	.338	.004592	47.9	.0305	.0325	30	20

201	C-T-J	1912	3	B	Orchard Mesa (timber flume), Colo	Rect.	12.0	1,349.0	63.3	16.77	3.80	1.114	------	0.000710	136.2	0.0114	0.0111	----	----
202								2,528.0	136.1	26.83	5.09	1.624	------	.000883	137.2	.0120	.0120	----	----
203								1,496.0	139.5	27.54	5.08	1.659	------	.000713	149.7	.0112	.0108	----	----
204								2,716.0	169.5	30.59	5.56	1.798	------	.000924	138.5	.0121	.0120	----	----
205								1,129.0	177.8	34.51	5.16	1.944	------	.000670	145.2	.0117	.0116	----	----
206	C-T-J	1912	3	B	Same, Near Grand Junction [7]	Rect.	12.0	2,708.0	209.3	35.21	5.96	1.964	------	.000965	138.9	.0123	.0120	----	----
207								1,129.0	213.1	43.17	4.94	2.249	------	.000555	142.7	.0123	.0122	----	----
208								1,360.0	214.9	38.04	5.66	2.074	------	.000779	143.8	.0120	.0119	----	----
209								2,672.0	219.4	36.14	6.08	2.002	------	.001079	134.7	.0126	.0128	----	----
210								1,496.0	228.8	39.45	5.81	2.124	------	.000858	137.6	.0126	.0123	----	----
211	B-11	1911	1	B	King-Hill project, Idaho	Rect.	6.5	1,119.0	86.99	15.02	5.79	1.35	------	.001288	139.2	.0115	.0113	----	----
212a	JDG	1906	New	B	Fleish flume, Calif., surfaced, grooved	Rect.	10.07	500.0	313.5	61.23	5.12	2.754	------	.000426	149.5	.0117	.0118	----	----
213	S-34	1913	8	A	Central Oregon Irrigation Co. no. 1, Flume, Oreg.	Rect.	16.0	1,100.0	399.7	44.13	9.06	1.90	2.317	.002106	143.1	.0118	.0116	----	----
214a	JDG	1906	10	C	Floriston flume, Calif., surfaced	Rect.	10.23	300.0	242.8	55.57	4.37	2.427	------	.000397	140.8	.0125	.0122	----	----
215a	JDG	1906	10	C	Same, lumber, battens rough	Rect.	10.23	300.0	403.5	71.79	5.62	2.727	------	.00064	134.5	.0130	.0131	----	----
216a	JDG	1906	10	B	do	Rect.	10.23	600.0	420.8	77.21	5.45	2.819	------	.00062	130.4	.0135	.0136	----	----
217	H-37	1913	23	B	Bitter Root Irrigation Co., Mont	Rect.	15.75	1,000.0	95.50	36.62	2.61	1.80	.239	.000239	125.9	.0131	.0130	----	----
218	H-32	1913	23	B	do	Rect.	17.7	1,000.0	142.10	53.10	2.68	2.24	.202	.000202	125.8	.0136	.0135	----	----
219	S-313	1923	4	A	Logan flume, Utah Power & Light Co., Utah.	Rect.	6.0	1,014.0	177.5	24.12	7.36	1.766	1.996	.001969	124.8	.0133	.0131	----	----
220	S-35c	1913	3	B	Arnold flume curves and tangent, Oreg			616.0	64.02	17.39	3.68	1.02	.647	.001050	112.6	.0134	.0132	----	----
221	S-35d	1913	3	A	Same, curves and tangents, Oreg	Rect.	11.9	2,200.0	64.02	18.64	3.43	1.08	2.307	.001049	102.1	.0148	.0151	----	----
222	S-35a	1913	3	C	Same, curve, Oreg			437.0	64.02	19.16	3.34	1.10	.452	.001034	98.9	.0153	.0153	----	----
223	S-35b	1913	3	B	Same, tangent, Oreg			1,147.0	64.02	19.18	3.34	1.10	1.208	.001053	97.9	.0154	.0154	----	----
224	C-T-J	1913	23	A	Oxford Canal, Fowler, Colo	Rect.	9.7	1,712.0	16.01	10.27	1.56	.84	.420	.000245	108.3	.0135	.0133	----	----
225	S-16	1913	23	B	Telluride flume, Utah	Rect.	5.9	400.0	159.3	25.65	6.21	1.67	.676	.001690	116.8	.0141	.0139	----	----
226a	Densmore	1928	23	B	Kern River, Borel Canal, Southern California Edison Co.	Rect.	10.2	1,647.9	408.0	57.33	7.12	2.66	2.089	.001268	122.5	.0144	.0143	----	----
227	S-356	1926	----	B	Cove flume, Utah Power & Light Co., Idaho.	Rect.	20.4	2,499.5	1,104.6	224.6	4.92	5.30	.641	.000256	133.6	.0145	.0147	----	----
228	S-33a	1913	----	C	Swalley-tangent, Oreg.			195.0	50.31	13.92	3.61	1.00	.288	.001479	93.8	.0157	.0159	----	----
229	S-33b	1913	----	C	Swalley, tangent and curve, Oreg	Rect.	10.0	159.0	50.31	12.34	4.08	.90	.308	.001934	97.5	.0149	.0150	----	----
230	S-33c	1913	----	C	do			244.0	50.31	11.56	4.35	.87	.524	.002145	99.3	.0157	.0146	----	----
231	S-33d	1913	----	B	Swalley, tangent and curves, Oreg			598.0	50.31	12.57	4.00	.93	1.119	.001871	95.9	.0153	.0153	----	----
232	S-314	1923	3	A	Olmstead flume, Utah Light & Power Co., Utah.	Rect.	10.0	954.5	419.20	59.18	7.09	2.793	1.074	.001125	126.8	.0140	.0140	----	----
233	S-340	1924	3	A	Hat Creek flume, Pacific Gas & Electric Co., Calif.	Rect.	16.3	2,500.0	368.8	102.76	3.59	3.556	1.142	.0004568	89.1	.0206	.0206	----	----
234a	K-1				Same, Aug. 21, 1922				427.0	113.5	3.76	3.754	.1760	.0007084	72.8	.0255	.0267	----	----
235a	K-2				Same, Aug. 23, 1922				422.0	114.1	3.70	3.768	.1742	.0007015	72.0	.0258	.0258	----	----
236a	K-3				Same, Aug. 24, 1922				440.0	115.4	3.81	3.788	.1619	.000652	76.8	.0242	.0242	----	----
237a	K-4	1922	1	B	Same, Aug. 25, 1922				442.0	115.0	3.84	3.780	.168	.0006762	76.2	.0244	.0244	----	----
238a	K-5				Same, Aug. 29, 1922	Rect.	16.3	248.3	434.0	114.3	3.80	3.768	.162	.000653	76.6	.0242	.0242	----	----
239a	K-6				Same, Sept. 1, 1922				421.0	111.7	3.77	3.720	.151	.000614	78.8	.0235	.0234	----	----
240a	K-7				Same, Sept. 18, 1922				108.7	90.75	1.20	3.308	.0242	.0000973	66.9	.0271	.0271	----	----
241a	K-8				do				456.1	115.5	3.85	3.79	.1521	.000612	82.2	.0227	.0232	----	----
242a	K-9	1922	1	C	Same, Nov. 5, 1922	Rect.	16.3	248.3	553.0	106.4	5.20	3.625	.126	.000506	121.6	.0152	.0152	----	----
243a	G-9	1926	5	B	Same, Jan. 25, 1926, no algae	Rect.	16.3	248.3	433.0	98.5	4.40	3.47	.1036	.000417	115.3	.0158	.0158	----	----
244a	G-10				Same, Jan. 26, 1926, no algae				416.0	96.0	4.33	3.42	.0995	.000400	117.6	.0156	.0156	----	----

[7] Of surfaced lumber, tongue and grooved joints. This flume superseded by no. 18 (concrete flumes).

TABLE 2.—*Elements of experiments on flumes for the determination of the coefficient of flow in Chezy's formula and retardation factors in Kutter's and Manning's formulas*—Continued

SHORT WOOD-PLANK FLUMES

Reference number	Experimenter and series	Year of test	Age, approximate	Test rating	Name, location, and condition of flume or canal	Shape	Width or diameter	Length of reach	Mean elements, water prism						Coefficients			Temperatures	
								L	Flow Q	Area A	Velocity V	Hydraulic radius R	Retardation loss h_f	Energy slope $S=\frac{h_f}{L}$	Chezy C	Kutter n	Manning n'	Air °C.	Water °C.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
			Yrs.				*Feet*	*Feet*	*Sec.-ft.*	*Sq. ft.*	*Ft. per sec.*	*Feet*	*Feet*	*Ft. per ft.*					
245	S-50	1913	22	B	Reno Light & Power Co. flume, Nev	Rect.	9.6	800.0	251.8	56.11	4.49	2.28	0.262	0.000327	164.1	0.0106	0.0104	-----	-----
246	H-16	1913	9	B	Alkali Creek flume, Mont	Rect.	10.0	500.0	88.74	16.42	5.40	1.23	.891	.001782	115.3	.0136	.0134	-----	-----
247					----do			700.0	153.63	21.17	7.26	1.48	1.416	.002023	132.5	.0123	.0120	-----	-----
248					Same, rough lumber, waterworn			700.0	160.8	21.71	7.41	1.51	1.445	.002064	132.6	.0123	.0120	-----	-----
249					Same, slight deposits, slime and moss			700.0	206.9	26.28	7.87	1.741	1.399	.001999	133.4	.1025	.0122	-----	-----
250	H-28	1913	New	C	Hedge Canal, Mont	Rect.	9.6	150.0	66.50	31.90	2.08	1.96	-------	.000133	129.0	.0129	.0129	-----	-----
251	H-30	1913	23	B	Bitter Root Valley Irrigation Co., Mont	Rect.	17.7	500.0	142.10	44.36	3.20	1.953	.159	.000318	128.5	.0131	.0129	-----	-----
252	S-329a	1923	-----	C	Lateral 2, flume 6a, Columbia Irrigation District, Wash.	Rect.	4.8	400.8	48.16	13.64	3.53	1.303	.264	.000659	120.5	.0131	.0129	-----	-----
253	H-11	1913	-----	B	Billings Land and Irrigation Co., Lateral 4, Mont.	Rect.	2.0	606.0	2.16	1.32	1.64	.40	.525	.000868	88.3	.0138	.0143	-----	-----
255a	USBR-F10	1915	-----	B	South Canal, Uncompahgre project, Colo.	Rect.	12.14	400.0	437.0	34.84	12.54	1.95	-------	.00560	120.0	.0141	.0139	-----	-----
256	SF-23	1897	9	C	Bear River, Utah	Rect.	-----	125.0	64.34	11.78	5.46	.87	-------	.0032	103.4	.0142	.0141	-----	-----
258	H-39	1913	23	B	Bitter Root Irrigation Co., Mont	Rect.	15.8	700.0	92.81	39.50	2.35	1.90	.174	.0002486	108.2	.0153	.0153	-----	-----
259	H-24	1913	14	B	Hedge Canal, old, surfaced, Mont	Rect.	7.6	1,000.0	65.20	21.30	3.06	1.62	.522	.000522	105.3	.0154	.0153	-----	-----
260	H-22	1913	14	C	----do	Rect.	7.8	738.0	69.07	22.19	3.11	1.65	.400	.000542	104.2	.0155	.0155	-----	-----
261	S-58	1913	-----	C	Dry Creek flume, Modesto Irrigation District, Calif.	Rect.	13.7	470.0	43.05	39.91	1.08	1.94	.027	.000057	102.3	.0156	.0162	-----	-----
262	S-61	1913	-----	C	Golden Rock Lower, Calif	Rect.	5.9	163.2	6.97	5.38	1.30	.66	.059	.0003615	84.2	.0157	.0165	-----	-----
263	S-351	1924	10	C	No. 1 flume, East Branch Canal, Canadian Pacific Railway project, Alberta, Canada.	Rect.	20.0	440.0	702.0	57.3	12.25	2.205	2.539	.005771	108.6	.0157	.0160	-----	-----

264	S-339	1924	-----	B	Santa Ana Wash flume, Riverside, Calif.	Rect.	8. 08	1, 100. 0	76. 12	23. 45	3. 25	1. 687	. 669	. 0006082	101. 3	. 0160	. 0160	-----	-----
265	C-T-J	1913	20	C	Flume no. 42, Willcox Canal, Colo.	Rect.	3. 5	360. 0	6. 33	3. 74	1. 69	. 685	-------	. 000661	79. 5	. 0168	. 0168	-----	-----
266	S-48	1913	-----	B	Wheeler, battens, worn, sand, Nev.	Rect.	6. 5	1, 014. 4	19. 35	12. 09	1. 61	1. 04	. 324	. 000319	88. 3	. 0164	. 0169	-----	-----
267	S-6	1913	Old	C	Lateral, Salt Lake City and Jordan, Utah.	Rect.	4. 5	400. 0	21. 96	3. 59	6. 11	. 59	4. 276	. 010690	77. 1	. 0170	. 0163	-----	-----
268	S-357	1926	21	B	Flume no. 2, Central Oregon Irrigation District, Bend, Oreg.	Rect.	13. 2	650. 0	398. 2	56. 02	7. 11	2. 58	1. 243	. 001912	101. 2	. 0173	. 0168	-----	-----
269a	Densmore	1928	23	C	Rich Gulch, Borel Canal, Southern California Edison Co.	Rect.	16. 0	513. 0	435. 0	107. 2	4. 06	3. 65	. 231	. 000450	100. 0	. 0174	. 0184	-----	-----
270	SF 45	1897	-----	C	Elm farm, projecting calking, Utah	Rect.	------	-------	. 97	1. 49	. 65	. 33	-------	. 00038	57. 6	. 0184	. 0214	-----	-----
271	S-76	1913	-----	B	Lower flume, Riverside, Calif.	Rect.	7. 6	746. 3	23. 64	6. 51	3. 63	. 70	2. 915	. 003906	69. 3	. 0189	. 0202	-----	-----
272	WBG-3	1913	14	C	Roller flume, slimed, Louisiana	Rect.	13. 0	500. 0	67. 25	55. 30	1. 22	2. 30	-------	. 000080	87. 5	. 0191	. 0196	-----	-----
273	SF-8	1897	7	C	Bear River flume, rocks on bottom, Utah.	Rect.	------	150. 0	197. 5	66. 60	2. 97	2. 86	-------	. 0004	87. 7	. 0201	. 0203	-----	-----
274	S-65	1913	-----	B	Fullerton, Calif.	Rect.	9. 8	672. 1	15. 72	10. 17	1. 55	. 86	. 423	. 000629	66. 6	. 0205	. 0218	-----	-----
275	SF-10	1897	7	C	Bear River, gravel, Utah	Rect.	------	100. 0	207. 0	84. 05	2. 46	2. 94	-------	. 00031	81. 8	. 0217	. 0219	-----	-----

LONG WOOD-STAVE FLUMES

300	W.B.D	1898	New	B	Provo Canyon Flume, Utah	Circ.	8. 5	[8]	73. 40	13. 36	5. 50	1. 45	-------	0. 001	143. 4	0. 0114	0. 0110	-----	-----
301	S-301A	1923	New	A	Central Oregon Irrigation District, Oreg., clean.	Circ.	12. 0	2, 650. 0	429. 00	41. 05	10. 45	2. 52	5. 30	. 002000	147. 2	. 0119	. 0118	30	15
302	S-301a	1923	New	A	Same, part of reach above. $V>V_c$	Circ.	12. 0	1, 200. 0	429. 00	39. 00	11. 00	2. 45	2. 43	. 002025	156. 2	. 0112	. 0111	30	15
303	S-301B	1926	3	A	Same as 301, 3 years later, algae	Circ.	12. 0	2, 650. 0	417. 50	45. 00	9. 28	2. 667	5. 23	. 001974	127. 9	. 0138	. 0137	35	15
304	S-301C	1928	5	A	Same as 301, 5 years later, algae	Circ.	12. 0	2, 650. 0	440. 00	45. 25	9. 72	2. 67	5. 25	. 001981	133. 7	. 0134	. 0131	-----	-----
305	S-301D	1931	8	A	Same as 301, 8 years later, algae	Circ.	12. 0	2, 650. 0	383. 80	41. 15	9. 33	2. 56	5. 03	. 001898	133. 8	. 0131	. 0130	-----	-----
306	S-328	1923	3	A	No. 1 main, Columbia Irrigation District, Wash.	Circ.	11. 5	2, 680. 0	246. 62	48. 64	5. 07	2. 794	1. 255	. 000468	140. 2	. 0127	. 0126	-----	-----
307	S-330A	1921	1	A	Selah-Moxee main flume, Wash	Circ.	6. 3	1, 500. 0	55. 41	14. 54	3. 81	1. 522	. 949	. 000633	122. 8	. 0132	. 0130	26	20
308	S-330B	1923	3	A	Same, 2 years later, just cleaned	Circ.	6. 3	1, 000. 0	65. 87	16. 49	3. 99	1. 627	. 540	. 000540	134. 8	. 0122	. 0122	-----	-----

SHORT WOOD-STAVE FLUMES

320	S-329	1923	3	B	No. 2, lateral 2, Columbia Irrigation District, Wash.	Circ.	6. 0	297. 0	48. 16	12. 32	3. 91	1. 400	0. 157	0. 000529	143. 8	0. 0113	0. 0109	-----	-----
321	S-326	1923	3	B	No. 3 main. Columbia Irrigation District, Wash.	Circ.	8. 9	368. 3	182. 9	30. 76	5. 95	2. 230	. 253	. 000687	151. 9	. 0114	. 0112	-----	-----
322	S-310	1923	3	B	McEathren flume, King Hill project, Idaho.	Circ.	8. 83	444. 1	193. 6	27. 20	7. 12	2. 068	. 497	. 001119	147. 9	. 0115	. 0113	25	17
323	S-333	1923	-----	B	Tieton unit, Yakima project, U.S.B.R.	Circ.	2. 33	905. 0	2. 20	1. 27	1. 73	. 431	. 518	. 000572	110. 1	. 0117	. 0117	32	20
324	S-332a	1923	-----	B	Nachez-Selah Irrigation District, Wash.	Circ.	7. 8	353. 0	112. 50	20. 50	5. 49	1. 801	. 295	. 000836	141. 6	. 0119	. 0116	33	24
325	S-332b	1923	-----	C	Same, sharp curve	Circ.	7. 8	97. 8	112. 50	20. 15	5. 58	1. 784	. 096	. 000982	133. 4	. 0125	. 0123	33	24
326	S-334	1923	New	B	Fruitvale-Schanno Ditch, Wash.	Circ.	4. 0	313. 0	23. 12	6. 79	3. 40	1. 036	. 263	. 000840	115. 4	. 0139	. 0130	33	21

[8] 1 mile.

TABLE 2.—*Elements of experiments on flumes for the determination of the coefficient of flow in Chezy's formula and retardation factors in Kutter's and Manning's formulas*—Continued

CONCRETE FLUME CHUTES OR INCLINED DROPS, COMPUTED FOR MEASURED CROSS SECTIONS AND MEASURED VELOCITIES

									Mean elements, water prism						Coefficients			Temperatures	
Reference number	Experimenter and series	Year of test	Age, approximate	Test rating	Name, location, and condition of flume or canal	Shape	Width or diameter	Length of reach L	Flow Q	Area A	Velocity V	Hydraulic radius R	Retardation loss h_f	Energy slope $S=\frac{h_f}{L}$	Chezy C	Kutter n	Manning n'	Air °C.	Water °C.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
			Yrs.				*Feet*	*Feet*	*Sec.-ft.*	*Sq. ft.*	*Ft. per sec.*	*Feet*	*Feet*	*Ft. per ft.*					
401a	USBR S-98	1911	2	---	Lizard chute no. 1, Boise project, Idaho	Trap.	3.00	250.0	0.44	0.12	4.97	0.039	20.6	0.08244	87.6	0.0087	0.0099	-----	-----
402a	USBR S-99 }	1911	2	---	{Same, 905 feet long with drop of 72 feet. Designed for 36 second-feet.	}Trap.	3.02	250.0 {	2.04	.33	9.33	.103	20.6	.08244	101.2	.0114	.0100	-----	-----
403a	USBR S-100 }								2.80	.38	10.90	.115	20.6	.08244	112.0	.0092	.0092	-----	-----
404a	USBR S-101	1911	2	---	Same. Side slopes 1:6	Trap.	3.03	250.0	5.72	.52	14.30	.164	20.6	.08244	123.2	.0091	.0089	-----	-----
405a	USBR S-102	1911	2	---	do	Trap.	3.04	250.0	7.97	.73	14.20	.208	20.6	.08244	108.2	.0106	.0106	-----	-----
406a	USBR S-103	1911	2	---	do	Trap.	3.06	250.0	16.40	1.10	19.20	.293	20.6	.08244	123.2	.0102	.0098	-----	-----
407a	USBR S-104	1911	2	---	Lizard chute no. 2, Boise project, Idaho	Trap.	3.00	40.0	.44	.15	6.90	.046	7.9	.1985	72.2	.0103	.0123	-----	-----
408a	USBR S-105	1911	2	---	do	Trap.	3.01	40.0	2.04	.30	7.84	.094	7.9	.1985	57.4	.0143	.0175	-----	-----
409a	USBR S-106	1911	2	---	do	Trap.	3.02	40.0	2.80	.32	11.10	.098	7.9	.1985	79.6	.0114	.0127	-----	-----
410a	USBR S-107	1911	2	---	Same, side slopes 1:6	Trap.	3.03	40.0	5.72	.48	19.80	.141	7.9	.1985	118.5	.0092	.0090	-----	-----
411a	USBR S-108	1911	2	---	do	Trap.	3.05	40.0	7.97	.63	16.00	.184	7.9	.1985	83.7	.0126	.0134	-----	-----
412a	USBR S-109	1911	2	---	do	Trap.	3.08	40.0	16.40	.94	23.80	.257	7.9	.1985	105.4	.0112	.0112	-----	-----
413a	USBR S-93	1911	2	---	Mora chute no. 1, Boise project, Idaho	Trap.	5.00	--------	27.50	1.60	22.02	.275	-------	.0810	147.5	.0087	.0081	-----	-----
414a	USBR S-96	1911	2	---	Arena chute, section 2, Boise project, Idaho.	Trap.	6.07	--------	23.30	1.34	19.60	.206	-------	.1530	110.5	.0104	.0103	-----	-----
415a	USBR S-97	1911	2	---	Arena chute; Boise project, Idaho	Trap.	6.00	--------	50.40	1.89	29.40	.285	-------	.2100	120.2	.0103	.0100	-----	-----
416a	USBR S-95	1911	2	---	Arena chute, section 1, Boise project, Idaho.	Trap.	6.08	--------	23.30	1.43	20.40	.219	-------	.2056	96.2	.0117	.0120	-----	-----
417a	USBR S-94	1911	2	---	Valley Mound Canal	Trap.	5.00	--------	22.40	1.35	26.30	.215	-------	.1585	142.7	.0086	.0081	-----	-----

METAL FLUME CHUTES OR INCLINED DROPS, COMPUTED FOR MEASURED CROSS SECTIONS AND MEASURED VELOCITIES

440	S-349B	1924	8	B	Hammerhill flume, Canadian Pacific Ry. project, Alberta, Canada.	96G	5.10	1,500.0	61.0	2.61	27.5	0.573	85.85	0.057231	152.1	0.0095	0.0089	----	----
441	S-349A	1924	8	A	do	96G	5.1	1,500.0	26.5	1.62	18.6	.433	86.29	.057527	118.2	.0113	.0109	----	----
442	S-345A	1924	9	B	Dalroy flume, Canadian Pacific Ry. project, Alberta, Canada.	192G	10.2	370.3	59.2	7.16	15.2	.993	11.91	.032164	85.0	.0173	.0174	----	----
443	S-345B	1924	9	A	do	192G	10.2	370.3	151.4	11.38	17.6	1.311	11.06	.035991	81.0	.0190	.0192	----	----

WOOD-PLANK FLUME CHUTES OR INCLINED DROPS, COMPUTED FOR MEASURED CROSS SECTIONS AND MEASURED VELOCITIES

450	S-346	1924	----	B	Lateral C-11 flume, Canadian Pacific Ry. project, Alberta, Canada.	Rect.	0.9	598.2	1.24	0.127	9.82	0.107	31.34	0.05238	131.5	0.0080	0.0078	----	----
451	S-347	1924	----	B	Secondary canal, Candian Pacific Ry. project, Alberta, Canada.	Rect.	4.18	516.0	6.16	.70	9.25	.155	12.80	.0248	149.4	.0078	.0092	----	----
452a	USBR S-90	1911	2	---	Fargo drop	Rect.	6.1	-------	95.0	3.48	32.3	.481	-------	.1250	131.6	.0105	.0100	----	----

CONCRETE CHUTES COMPUTED FOR MEASURED SECTIONS BUT WITH VELOCITY, V, DETERMINED AS Q/A

401′a	USBR S-98	1911	2	B	Lizard chute no. 1 (see above)	Trap.	3.00	250.0	0.44	0.12	3.67	0.039	20.6	0.08244	64.76	0.0107	0.0105	----	----
402′a	USBR S-99	1911	2	B	do	Trap.	3.02	250.0	2.04	.33	6.18	.103	20.6	.08244	67.25	.0130	.0150	----	----
403′a	USBR S-100	1911	2	B	do	Trap.	3.02	250.0	2.80	.38	7.47	.115	20.6	.08244	76.56	.0120	.0120	----	----
404′a	USBR S-101	1911	2	B	do	Trap.	3.03	250.0	5.72	.52	10.46	.164	20.6	.08244	90.11	.0116	.0115	----	----
405′a	USBR S-102	1911	2	B	do	Trap.	3.04	250.0	7.97	.73	10.99	.208	20.6	.08244	83.87	.0129	.0129	----	----
406′a	USBR S-103	1911	2	B	do	Trap.	3.06	250.0	16.42	1.10	15.05	.293	20.6	.08244	97.42	.0124	.0124	----	----
407′a	USBR S-104	1911	2	C	Lizard chute no. 2 (see above)	Trap.	3.00	40.0	.44	.15	3.12	.046	7.9	.19850	32.80	.0175	.0176	----	----
408′a	USBR S-105	1911	2	C	do	Trap.	3.01	40.0	2.04	.30	6.80	.094	7.9	.19850	49.85	.0158	.0164	----	----
409′a	USBR S-106	1911	2	C	do	Trap.	3.02	40.0	2.80	.32	8.89	.098	7.9	.19850	63.72	.0134	.0134	----	----
410′a	USBR S-107	1911	2	C	do	Trap.	3.03	40.0	5.72	.48	12.25	.141	7.9	.19850	73.35	.0131	.0131	----	----
411′a	USBR S-108	1911	2	C	do	Trap.	3.05	40.0	7.97	.63	12.65	.184	7.9	.19850	66.13	.0149	.0150	----	----
412′a	USBR S-109	1911	2	C	do	Trap.	3.08	40.0	16.42	.94	17.66	.257	7.9	.19850	78.18	.0142	.0142	----	----
413′a	USBR S-93	1911	2	B	Mora chute no. 1	Trap.	5.0	-------	27.50	1.60	17.69	.275	-------	.08100	118.49	.0104	.0100	----	----
414′a	USBR S-96	1911	2	B	Arena chute, section 2	Rect.	6.07	-------	23.3	1.34	17.49	.206	-------	.15300	98.41	.0104	.0100	----	----
415′a	USBR S-97	1911	2	B	do	Rect.	6.0	-------	50.40	1.89	26.67	.285	-------	.21000	109.0	.0112	.0109	----	----
416′a	USBR S-95	1911	2	B	Arena chute, section 1	Rect.	6.08	-------	23.3	1.43	16.38	.219	-------	.20560	77.16	.0139	.0149	----	----
417′a	USBR S-94	1911	2	B	Valley Mound canal	Trap.	5.0	-------	22.35	1.35	18.77	.215	-------	.15850	101.85	.0112	.0109	----	----
418′	C-T-J	1912	2	B	Long Pond chute	Rect.	4.4	600.0	35.79	2.78	12.87	.489	17.9	.02978	106.6	.0125	.0124	----	----
419′	C-T-J	1912	2	B	do	Rect.	4.4	600.0	78.32	4.29	18.26	.674	17.8	.02968	128.9	.0113	.0111	----	----
420′	C-T-J	1912	2	B	do	Rect.	4.4	600.0	100.38	5.61	17.89	.806	17.65	.02943	116.1	.0128	.0127	----	----
421′	C-T-J	1912	2	B	do	Rect.	4.4	600.0	104.47	5.17	20.21	.765	17.9	.02990	133.5	.0111	.0108	----	----
422′	C-T-J	1912	2	B	do	Rect.	4.4	600.0	122.94	6.24	19.70	.862	17.8	.02971	122.9	.0123	.0120	----	----
423′	C-T-J	1912	6	B	Dry Creek flume	Rect.	7.9	514.5	154.00	9.76	15.78	.942	7.5	.01459	134.5	.0115	.0110	----	----

TABLE 2.—*Elements of experiments on flumes for the determination of the coefficient of flow in Chezy's formula and retardation factors in Kutter's, and Manning's formulas*—Continued

CONCRETE CHUTES COMPUTED FOR MEASURED SECTIONS BUT WITH VELOCITY, *V*, DETERMINED AS *Q/A*—Continued

Reference number	Experimenter and series	Year of test	Age, approximate	Test rating	Name, location, and condition of flume or canal	Shape	Width or diameter	Length of reach L	Mean elements, water prism: Flow Q	Area A	Velocity V	Hydraulic radius R	Retardation loss h_f	Energy slope $S=\frac{h_f}{L}$	Coefficients: Chezy C	Kutter n	Manning n'	Temperatures: Air °C.	Water °C.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
			Yrs.				*Feet*	*Feet*	*Sec.-ft.*	*Sq. ft.*	*Ft. per sec.*	*Feet*	*Feet*	*Ft. per ft.*					
424'a	USBR	(9)	(10)	A	Sulphur Creek waste, Yakima project, Wash.	Circ.	8	900.0	52.5	-----	12.4	0.69	18.46	0.0205	104.3	0.0130	0.0134	-----	-----
425'a	USBR	(9)	(10)	A	Same reach, on 2° curve	Circ.	8	900.0	242.0	-----	19.1	1.36	18.57	.0206	114.1	.0140	.0137	-----	-----
426'a	USBR	(9)	(10)	A	Same reach, no. 4 of U.S. Bureau of Reclamation records.	Circ.	8	900.0	247.0	-----	19.3	1.37	18.61	.0207	114.4	.0140	.0137	-----	-----
427'a	USBR	(9)	(10)	A	Same reach, no. 6, on tangent	Circ.	8	1,300.0	45.0	-----	12.5	.62	18.85	.0145	131.7	.0109	.0104	-----	-----
428'a	USBR	(9)	(10)	A	Same reach, concrete deposited against	Circ.	8	1,300.0	52.5	-----	13.1	.67	18.80	.0144	133.1	.0109	.0104	-----	-----
429'a	USBR	(9)	(10)	A	Same reach, wood forms, without	Circ.	8	1,300.0	242.0	-----	20.6	1.30	18.79	.0144	150.5	.0108	.0103	-----	-----
430'a	USBR	(9)	(10)	A	Same reach, retouching surface	Circ.	8	1,300.0	247.0	-----	20.4	1.33	18.71	.0144	147.8	.0110	.0106	-----	-----

METAL CHUTES COMPUTED FOR MEASURED SECTIONS BUT WITH VELOCITY, *V*, DETERMINED AS *Q/A*

Reference number	Experimenter and series	Year of test	Age, approximate	Test rating	Name, location, and condition of flume or canal	Shape	Width or diameter	Length of reach L	Flow Q	Area A	Velocity V	Hydraulic radius R	Retardation loss h_f	Energy slope S	Chezy C	Kutter n	Manning n'	Air °C.	Water °C.
440'	S-349B	1924	8	A	Hammerhill flume, Canadian Pacific Railway project, Alberta, Canada.	96G	5.1	1,500.0	61.0	2.61	23.4	0.573	85.74	0.05716	129.1	0.0109	0.0104	-----	-----
441'	S-349A	1924	8	A	do	96G	5.1	1,500.0	26.5	1.62	16.4	.433	86.08	.05738	103.8	.0125	.0125	-----	-----
442'	S-345A	1924	9	B	Dalroy flume, Canadian Pacific Railway project, Alberta, Canada.	192G	10.2	370.3	59.2	7.16	8.27	.993	11.91	.032164	46.3	.0294	.0321	-----	-----
443'	S-345B	1924	9	A	do	192G	10.2	370.3	151.4	11.38	13.30	1.311	11.06	.035991	61.2	.0245	.0254	-----	-----

WOOD-PLANK CHUTES COMPUTED FOR MEASURED SECTIONS BUT WITH VELOCITY, V, DETERMINED AS Q/A

452'a	USBR S-91	1911	2	B	Fargo drop	Rect.	6.1	--------	95.0	3.48	27.61	0.481	--------	0.1250	112.6	0.0119	0.0118	-----	-----

CONCRETE CHUTES COMPUTED FOR MEASURED VELOCITY, V, AND SECTION COMPUTED AS WITHOUT ENTRAINED AIR, BY $A=Q/V$

401''a	USBR S-115	1911	2	B	Lizard chute no. 1 (see above)	Rect.	3.00	250.0	0.44	0.09	4.97	0.029	20.6	0.08244	102.0	0.0074	0.0071	-----	-----
402''a	USBR S-116	1911	2	B	do	Rect.	3.01	250.0	2.04	.21	9.33	.070	20.6	.08244	123.2	.0078	.0075	-----	-----
403''a	USBR S-117	1911	2	B	do	Rect.	3.01	250.0	2.80	.26	10.91	.081	20.6	.08244	133.6	.0076	.0073	-----	-----
404''a	USBR S-118	1911	2	B	do	Rect.	3.02	250.0	5.72	.41	14.32	.122	20.6	.08244	142.8	.0078	.0073	-----	-----
405''a	USBR S-119	1911	2	B	do	Rect.	3.03	250.0	7.97	.56	14.17	.166	20.6	.08244	121.0	.0093	.0091	-----	-----
406''a	USBR S-120	1911	2	B	do	Rect.	3.05	250.0	16.4	.86	19.15	.240	20.6	.08244	135.5	.0091	.0088	-----	-----
407''a	USBR S-121	1911	2	C	Lizard chute no. 2 (see above)	Rect.	3.00	40.0	.44	.06	6.90	.021	7.9	.19850	107.0	.0067	.0063	-----	-----
408''a	USBR S-122	1911	2	C	do	Rect.	3.01	40.0	2.04	.27	7.84	.082	7.9	.19850	61.5	.0129	.0129	-----	-----
409''a	USBR S-123	1911	2	C	do	Rect.	3.02	40.0	2.80	.25	11.11	.080	7.9	.19850	88.5	.0102	.0010	-----	-----
410''a	USBR S-124	1911	2	C	do	Rect.	3.03	40.0	5.72	.29	19.82	.091	7.9	.19850	147.9	.0072	.0070	-----	-----
411''a	USBR S-125	1911	2	C	do	Rect.	3.04	40.0	7.97	.50	16.00	.150	7.9	.19850	92.9	.0112	.0110	-----	-----
412''a	USBR S-126	1911	2	C	do	Rect.	3.06	40.0	16.4	.70	23.81	.199	7.9	.19850	119.7	.0098	.0096	-----	-----
413''a	USBR S-110	1911	2	B	Mora chute no. 1	Rect.	5.00	--------	27.50	1.25	22.02	.227	--------	.08100	162.3	.0078	.0072	-----	-----
414''a	USBR S-113	1911	2	B	Arena chute, section 2	Rect.	6.06	--------	23.3	1.18	19.61	.186	--------	.15300	116.2	.0099	.0097	-----	-----
415''a	USBR S-114	1911	2	B	Arena chute	Rect.	6.00	--------	50.40	1.71	29.41	.261	--------	.21000	125.6	.0098	.0096	-----	-----
416''a	USBR S-112	1911	2	B	Arena chute, section 1	Rect.	6.06	--------	23.3	1.15	20.41	.179	--------	.20560	106.4	.0106	.0093	-----	-----
417''a	USBR S-111	1911	2	B	Valley Mound chute	Rect.	5.00	--------	22.35	.85	26.34	.159	--------	.15850	169.2	.0072	.0066	-----	-----

WOOD-PLANK CHUTES COMPUTED FOR MEASURED VELOCITY, V, AND SECTION COMPUTED AS WITHOUT ENTRAINED AIR, BY $A=Q/V$

452''a	USBR S-92	1911	2	B	Fargo drop	Rect.	6.1	--------	95.0	3.00	32.26	0.422	--------	0.1250	140.5	0.0097	0.0092	-----	-----

[9] About 1912.
[10] Built 1911.

TABLE 3.—*Elements of experiments on flumes to determine the energy slope for use in solution of flow formulas. See table 2. Difference in elements at 2 ends of reach indicates net variation from uniform flow that exists in field practice*

LONG CONCRETE FLUMES

Reference no.	Used in design Q'	Observed Q	Elements, upper end of reach L					Length of reach L	Elements, lower end of reach L					Retardation loss		Constructed slope s
			Area a_1	Velocity v_1	Velocity head h_1	Elevations			Area a_2	Velocity v_2	Velocity head h_2	Elevations		In reach $E_1-E_2=h_f$	Per foot $S=\frac{hf}{L}$	
						Surface Z_1	Energy gradient $E_1=h_1+Z_1$					Surface Z_2	Energy gradient $E_2=h_2+Z_2$			
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
	Sec.-ft.	*Sec.-ft.*	*Sq. ft.*	*Ft. per sec.*	*Feet*	*Feet*	*Feet*	*Feet*	*Sq. ft.*	*Ft. per sec.*	*Feet*	*Feet*	*Feet*	*Feet*	*Ft. per ft.*	*Ft. per ft.*
1a	334.0	307.2	48.73	6.30	0.62	4,104.79	4,105.41	3,953.0	48.73	6.30	0.62	4,102.10	4,102.72	2.69	0.000680	----------
2a	334.0	133.2	27.61	4.82	.36	4,102.98	4,103.34	4,168.0	34.65	3.84	.23	4,100.81	4,101.04	2.30	.000552	----------
3a	334.0	40.8	13.75	2.97	.14	4,101.68	4,101.82	4,168.0	18.70	2.16	.07	4,099.32	4,099.39	2.42	.000582	----------
4a	334.0	210.2	37.73	5.57	.48	4,103.81	4,104.29	3,988.0	42.90	4.90	.37	4,101.49	4,101.86	2.43	.000609	----------
5a	334.0	99.6	23.76	4.23	.28	4,102.50	4,102.78	3,276.0	25.19	3.99	.25	4,100.42	4,100.67	2.11	.000644	----------
6a	334.0	83.1	21.12	3.95	.24	4,102.29	4,102.53	3,278.0	23.10	3.62	.20	4,100.26	4,100.46	2.07	.000631	----------
7a	334.0	178.5	35.20	5.07	.40	4,103.54	4,103.94	3,278.0	34.21	5.22	.42	4,101.24	4,101.66	2.28	.000695	----------
8a	334.0	124.7	28.60	4.36	.30	4,102.94	4,103.24	3,280.0	30.80	4.05	.26	4,100.93	4,101.19	2.05	.000625	----------
9a	334.0	154.2	32.01	4.82	.36	4,103.25	4,103.61	3,280.0	36.08	4.27	.28	4,101.41	4,101.69	1.92	.000684	----------
10a	300.0	274.0	----------	----------	----------	----------	----------	----------	----------	----------	----------	----------	----------	----------	----------	0.00165
11a	300.0	160.0	----------	----------	----------	----------	----------	----------	----------	----------	----------	----------	----------	----------	----------	.00165
12a	----------	40.8	13.75	2.97	.137	4,101.680	4,101.817	2,174.0	13.86	2.94	.135	4,100.210	4,100.345	1.472	.000677	----------
13	315	209.1	25.47	8.21	1.047	97.960	99.007	1,061.0	28.66	7.30	.828	96.260	97.088	1.919	.001808	.002
14	315	209.1	27.12	7.69	.919	98.150	99.069	1,206.0	27.85	7.50	.875	95.830	96.705	2.364	.001899	.002
15	45.0	33.3	6.29	5.30	.44	52.24	52.68	1,950.0	7.39	4.51	.31	48.50	48.81	3.87	.001985	.002
16	----------	42.88	11.71	3.66	.208	101.830	102.038	1,172.4	11.23	3.82	.227	100.691	100.918	1.120	.000955	----------
17	----------	44.4	12.06	3.68	.210	101.962	102.172	1,172.4	11.53	3.85	.230	100.812	101.042	1.130	.000964	----------
18	800.0	372.0	68.64	5.42	.457	4,774.39	4,774.85	1,700.0	68.81	5.41	.455	4,773.04	4,773.50	1.35	.0007941	.0008
19	800.0	372.0	68.81	5.41	.455	4,773.04	4,773.50	800.0	71.70	5.19	.419	4,772.57	4,772.99	.51	.0006375	.0008
20	800.0	372.0	71.70	5.19	.419	4,772.57	4,772.99	1,100.0	75.61	4.92	.376	4,771.92	4,772.30	.69	.0006273	.0008
21	----------	107.6	30.29	3.55	.20	88.71	88.91	3,000.0	27.96	3.85	.23	86.85	87.08	1.83	.000610	----------
22	----------	66.58	13.79	4.83	.363	114.468	114.830	1,100.0	15.95	4.17	.268	112.927	113.195	1.635	.001486	----------
23	913.0	217.0	62.15	3.49	.189	95.059	95.248	1,361.0	69.52	3.12	.151	94.747	94.898	.350	.000257	.000383
24	913.0	497.2	102.56	4.85	.366	97.169	97.535	1,361.0	107.99	4.60	.330	96.667	96.997	.538	.000395	.000383
25a	900.0	472.0	105.8	4.46	.309	94.570	94.88	2,660.0	107.6	4.38	.298	93.57	93.87	1.01	.000380	.0003802
26a	900.0	629.0	124.4	5.06	.398	95.47	95.87	2,660.0	124.8	5.04	.395	94.39	94.785	1.08	.000406	.0003802
27a	900.0	472.0	107.6	4.38	.30	93.57	93.87	3,000.0	102.3	4.61	.33	92.34	92.67	1.20	.000400	.0003802
28a	900.0	629.0	124.8	5.04	.39	94.39	94.78	3,000.0	115.6	5.44	.46	92.99	93.45	1.33	.000443	.0003802
29a	900.0	699.0	130.2	5.37	.45	94.65	95.10	3,000.0	120.4	5.80	.52	93.22	93.74	1.36	.000453	.0003802

30a	900.0	472.0	102.3	4.61	.33	92.34	92.67	1,702.0	103.3	4.57	.32	91.68	92.00	.67	.000394	.0003802
31a	900.0	629.0	115.6	5.44	.46	92.99	93.45	1,702.0	115.6	5.44	.46	92.28	92.74	.71	.000417	.0003802
32a	900.0	699.0	120.4	5.80	.52	93.22	93.74	1,702.0	119.2	5.86	.53	92.45	92.98	.76	.000446	.0003802
33a	900.0	472.0	103.3	4.57	.32	91.68	92.00	1,398.0	98.8	4.78	.35	91.07	91.42	.58	.000415	.0003802
34a	900.0	629.0	115.6	5.44	.46	92.28	92.74	1,398.0	107.0	5.86	.53	91.48	92.01	.73	.000522	.0003802
35a	900.0	699.0	119.2	5.86	.53	92.45	92.98	1,398.0	109.0	6.41	.64	91.58	92.22	.76	.000544	.0003802
36a	900.0	472.0	94.0	5.02	.39	90.86	91.25	1,120.0	91.4	5.16	.41	90.22	90.63	.62	.000554	.0003802
37a	900.0	629.0	99.3	6.33	.62	91.12	91.74	1,120.0	89.5	7.03	.77	90.12	90.89	.85	.000759	.0003802
38a	900.0	699.0	99.9	7.00	.76	91.15	91.91	1,120.0	88.6	7.89	.97	90.07	91.04	.87	.000777	.0003802
39	550.0	537.0	78.90	6.82	.723	3,700.665	3,701.388	1,100.0	77.80	6.92	.744	3,699.530	3,700.274	1.114	.001013	.000936
40	550.0	537.0	77.80	6.92	.744	3,699.530	3,700.274	1,632.8	65.30	8.22	1.050	3,697.460	3,698.510	1.764	.001081	.001
41	550.0	515.0	76.00	6.77	.713	3,674.160	3,674.873	1,125.0	75.80	6.79	.717	3,673.190	3,673.907	.966	.000859	.0008
42	550.0	515.0	75.85	6.79	.720	3,673.190	3,673.910	1,100.0	78.15	6.59	.680	3,672.450	3,673.130	.780	.000709	.0008
43	550.0	515.0	78.40	6.57	.671	3,672.425	3,673.096	1,671.4	77.60	6.64	.686	3,671.125	3,671.811	1.285	.000769	.0008
44	550.0	540.0	80.70	6.69	.700	3,673.400	3,674.100	1,075.0	78.80	6.85	.730	3,672.640	3,673.370	.730	.000679	.0008
45	550.0	540.0	80.20	6.74	.710	3,672.550	3,673.260	1,671.4	79.70	6.75	.710	3,671.260	3,671.970	1.290	.000772	.0008
46	550.0	102.5	23.22	4.41	.302	3,669.650	3,669.952	2,890.5	23.64	4.34	.293	3,667.350	3,667.643	2.309	.000799	.0008
47	550.0	220.0	39.38	5.59	.486	3,670.800	3,671.286	2,890.5	41.50	5.30	.438	3,668.620	3,669.058	2.228	.000771	.0008
48	550.0	322.0	51.09	6.30	.617	3,671.650	3,672.267	2,890.5	54.54	5.90	.541	3,669.550	3,670.091	2.176	.000753	.0008
49	550.0	516.0	66.80	7.74	.930	3,597.970	3,598.900	1,050.0	70.80	7.30	.830	3,597.320	3,598.150	.750	.000714	.0008
50	550.0	289.4	43.41	6.67	.692	3,596.100	3,596.792	750.0	45.61	6.35	.627	3,595.670	3,596.297	.495	.000660	.0008
51	550.0	516.0	70.80	7.30	.830	3,597.320	3,598.150	1,150.0	68.20	7.56	.890	3,596.180	3,597.07	1.080	.000939	.0008
52	550.0	289.4	45.61	6.35	.627	3,595.670	3,596.297	1,100.0	43.83	6.60	.677	3,594.640	3,595.317	.980	.000891	.0008
53	550.0	516.0	68.20	7.58	.890	3,596.180	3,597.070	1,100.0	68.60	7.53	.880	3,595.295	3,596.175	.895	.000809	.0008
54	550.0	289.4	43.83	6.60	.677	3,594.640	3,595.317	1,099.2	43.07	6.72	.702	3,593.655	3,594.357	.960	.000874	.0008

SHORT CONCRETE FLUMES

70	230.0	193.6	25.10	7.71	0.923	97.520	98.443	752.0	27.47	7.05	0.772	96.255	97.027	1.416	0.001883	0.0024
71	----------	129.1	27.09	4.767	.353	99.614	99.967	1,124.3	23.17	5.574	.483	98.449	98.932	1.035	.000921	.000815
72	----------	494.5	52.59	9.39	1.371	48.778	50.149	1,450.0	42.84	11.53	2.065	45.460	47.525	2.624	.001876	----------
73	----------	494.5	52.59	9.39	1.371	48.778	50.149	626.0	53.46	9.24	1.327	47.807	49.134	1.015	.001660	----------
74	----------	494.5	53.46	9.24	1.327	47.807	49.134	824.0	42.84	11.53	2.065	45.460	47.525	1.609	.002039	----------
75	----------	39.62	4.57	8.68	1.171	2.612	3,783	164.2	6.155	6.44	.645	1.936	2.581	1.202	.007320	.007036
76a	300.0	205.0	27.69	7.40	.86	648.30	649.16	640.0	30.11	6.81	.72	647.40	648.12	1.04	.001625	.00124
77a	300.0	205.0	30.11	6.81	.72	647.40	648.12	340.0	28.81	7.12	.79	646.60	647.39	.73	.002147	.00124
78a	300.0	205.0	28.81	7.12	.79	646.60	647.39	1,075.0	28.55	7.18	.80	644.70	645.50	1.89	.001758	.00124
79a	300.0	205.0	27.69	7.40	.86	648.30	649.16	2,055.0	28.55	7.18	.80	644.70	645.50	3.66	.001781	.00124
80	----------	30.13	8.87	3.40	.180	97.830	98.010	420.5	10.69	2.82	.124	97.500	97.624	.386	.000918	----------
81	1,200.0	247.0	90.88	2.72	.115	95.621	95.736	579.9	101.10	2.44	.093	95.457	95.550	.186	.0003208	----------

TABLE 3.—*Elements of experiments on flumes to determine the energy slope for use in solution of flow formulas. See table 2. Difference in elements at 2 ends of reach indicates net variation from uniform flow that exists in field practice*—Continued

LONG METAL FLUMES—FLUSH INTERIORS

Reference no.	Used in design Q'	Observed Q	Elements, upper end of reach L: Area a_1	Elements, upper end of reach L: Velocity v_1	Elements, upper end of reach L: Velocity head h_1	Elements, upper end of reach L: Elevations: Surface Z_1	Elements, upper end of reach L: Elevations: Energy gradient $E_1=h_1+Z_1$	Length of reach L	Elements, lower end of reach L: Area a_2	Elements, lower end of reach L: Velocity v_2	Elements, lower end of reach L: Velocity head h_2	Elements, lower end of reach L: Elevations: Surface Z_2	Elements, lower end of reach L: Elevations: Energy gradient $E_2=h_2+Z_2$	Retardation loss: In reach $E_1-E_2=h_f$	Retardation loss: Per foot $S=\frac{h_f}{L}$	Constructed slope s
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
	Sec.-ft.	*Sec.-ft.*	*Sq. ft.*	*Ft. per sec.*	*Feet*	*Feet*	*Feet*	*Feet*	*Sq. ft.*	*Ft. per sec.*	*Feet*	*Feet*	*Feet*	*Feet*	*Ft. per ft.*	*Ft. per ft.*
98		80.75	14.05	5.75	0.514	100.600	101.114	1,800.0	13.80	5.86	0.534	98.860	99.394	1.72	0.000956	
99		70.72	14.77	4.79	.357	71.132	71.489	1,806.0	15.35	4.61	.330	69.320	69.650	1.839	.001018	
100		19.6	3.90	5.02	.39	95.90	96.37	2,850.0	6.16	3.18	.16	89.16	89.32	7.05	.002474	
101		55.41	14.10	3.93	.240	100.930	101.170	1,500.0	14.70	3.77	.221	100.000	100.221	.949	.000633	
102		65.87	14.42	4.57	.325	98.916	99.241	995.7	15.10	4.36	.295	98.323	98.618	.623	.000625	
103	225.0	159.9	25.19	6.35	.627	100.324	100.951	1,800.0	25.30	6.32	.621	98.548	99.169	1.782	.000990	0.0006
104	386.0	314.0	56.37	5.57	.482	1,017.991	1,018.473	2,160.0	55.80	5.63	.493	1,016.657	1,017.150	1.323	.000613	0.0006
105		212.6	34.98	6.08	.575	98.788	99.363	1,113.0	34.80	6.11	.580	97.340	97.920	1.443	.00130	.0012
106		212.6	35.72	5.95	.550	97.350	99.910	1,150.0	36.64	5.80	.523	96.010	96.533	1.377	.00120	.0012
107		212.6	34.70	6.13	.584	94.940	95.524	1,700.0	32.31	6.58	.673	92.832	93.505	2.019	.00119	.0012
108		212.6	32.14	6.61	.68	92.31	92.99	3,370.0	33.78	6.29	.62	88.07	88.69	4.30	.00128	.0012
109		212.6	32.70	6.50	.657	87.780	88.437	3,651.0	27.47	7.74	.931	82.960	83.891	4.546	.00125	

LONG METAL FLUMES—PROJECTING BANDS

Reference no.	Used in design Q'	Observed Q	Area a_1	Velocity v_1	Velocity head h_1	Surface Z_1	Energy gradient E_1	Length of reach L	Area a_2	Velocity v_2	Velocity head h_2	Surface Z_2	Energy gradient E_2	In reach h_f	Per foot S	Constructed slope s
110		18.97	5.10	3.72	0.215	98.370	98.585	970.0	5.10	3.72	0.215	96.945	97.160	1.425	0.001469	
111		18.97	5.10	3.72	.215	98.370	98.585	229.0	4.91	3.86	.231	98.031	98.262	.323	.001411	
112		121.8	18.81	6.48	.652	20.570	21.222	1,003.0	19.34	6.30	.616	18.750	19.366	1.856	.001850	
113		74.9	20.96	3.57	.198	89.864	90.062	1,984.3	17.70	4.22	.277	88.171	88.448	1.614	.000814	
114		74.9	20.96	3.57	.198	89.864	90.062	565.0	24.00	3.12	.151	89.460	89.611	.451	.000798	
115		74.9	24.00	3.12	.151	89.460	89.611	1,419.3	17.70	4.20	.274	88.171	88.445	1.166	.000822	

120a	87.8	39.0	6.90	5.65	0.496	98.070	98.566	525.0	6.70	5.82	0.527	97.420	97.947	0.619	0.001179	0.0012
121a	87.8	14.5	3.15	4.61	.330	97.240	97.570	525.0	3.60	4.03	.252	96.700	96.952	.618	.001177	.0012
122a	149.0	56.0	7.90	7.09	.782	96.900	97.682	400.0	8.50	6.59	.675	96.500	97.175	.507	.001268	.0012
123a	149.0	45.1	7.80	5.78	.519	96.850	97.369	400.0	9.85	4.58	.326	96.650	96.976	.393	.000982	.0012
124	425.0	194.1	27.69	7.02	.786	92.978	93.764	262.7	28.52	6.81	.719	92.831	93.550	.214	.000815	.00111
125	425.0	226.2	29.80	7.59	.894	93.482	94.376	524.0	31.77	7.12	.788	93.162	93.950	.426	.000813	.00111
126	425.0	237.0	31.73	7.47	.865	93.653	94.518	524.0	32.44	7.31	.829	93.205	94.034	.484	.000924	.00111
127	----	11.02	4.09	2.69	.112	99.042	99.154	55.6	4.66	2.36	.087	99.044	99.131	.023	.000414	.0014
128a	230.0	78.0	11.80	6.61	.679	96.800	97.479	600.0	12.35	6.32	.621	96.050	96.671	.808	.001347	.0012
129a	230.0	95.0	13.90	6.84	.727	97.050	97.777	600.0	13.25	7.17	.799	96.200	96.999	.778	.001297	.0012
130a	230.0	64.0	10.40	6.16	.590	96.550	97.140	600.0	11.00	5.82	.527	95.950	96.477	.663	.001105	.0012
131a	230.0	67.7	10.90	6.21	.600	96.550	97.150	600.0	11.80	5.74	.512	95.950	96.462	.688	.001147	.0012
132a	230.0	44.3	8.90	4.98	.386	96.260	96.646	600.0	9.65	4.59	.328	95.620	95.948	.698	.001163	.0012
133	----	7.04	1.216	5.79	.521	100.000	100.521	200.0	1.12	6.26	.609	98.953	99.562	.959	.00480	----
134	----	7.04	1.086	6.48	.653	100.000	100.653	60.0	1.25	5.65	.496	99.584	100.080	.574	.00957	----
135	----	19.6	3.90	5.02	.392	95.900	96.292	450.0	3.35	5.85	.532	94.875	95.407	.885	.001966	----
136	----	19.6	3.98	4.92	.376	94.435	94.811	650.0	3.34	5.87	.536	92.930	93.466	1.345	.00207	----
137	----	19.6	4.01	4.89	.370	92.380	92.750	325.0	4.89	4.01	.250	91.675	91.935	.815	.00251	----
138a	116.0	40.0	8.15	4.91	.375	96.700	97.075	690.0	10.15	3.94	.241	96.220	96.461	.614	.000890	.0012
139a	116.0	18.1	5.00	3.62	.204	96.020	96.224	690.0	7.05	2.57	.103	95.700	95.803	.421	.000610	.0012
140	----	112.5	20.13	5.59	.486	99.200	99.686	963.0	19.17	5.87	.536	98.305	98.841	.845	.000878	----
141	465.0	259.0	37.54	6.90	.740	97.396	98.136	800.8	47.33	5.47	.465	97.137	97.602	.534	.000666	.0014
142a	----	36.5	----	----	----	96.07	----	350.0	----	----	----	96.00	----	----	----	----
143a	----	20.8	----	----	----	95.89	----	300.0	----	----	----	95.84	----	----	----	----
144a	----	62.5	----	----	----	97.46	----	197.0	----	----	----	96.54	----	----	----	----
145a	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----
146a	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----
147	----	227.5	39.79	5.72	.508	98.470	98.978	241.3	36.09	6.30	.617	98.114	98.731	.247	.001024	----
148	----	238.0	39.23	6.07	.572	98.460	99.032	241.3	36.48	6.53	.662	98.143	98.805	.227	.000941	----
149	----	224.9	35.68	6.30	.617	98.051	98.668	241.3	32.85	6.85	.729	97.742	98.471	.197	.000816	----
150	----	223.1	36.10	6.18	.594	97.996	98.590	241.3	32.74	6.81	.721	97.765	98.486	.104	.000431	----
151	----	14.67	4.40	3.34	.173	99.810	99.983	184.0	4.96	2.96	.136	99.709	99.845	.138	.000750	----
152	----	14.7	3.75	3,92	.238	97.165	97.403	139.3	3.87	3.80	.224	97.038	97.262	.142	.001019	----
153	----	19.16	5.70	3.36	.175	96.382	96.557	302.3	5.44	3.52	.192	95.712	95.904	.653	.00216	----
154	----	20.39	6.90	2.96	.136	92.568	92.704	92.0	6.50	3.14	.153	92.494	92.647	.057	.000620	----
155	----	8.36	1.36	6.14	.586	30.310	30.896	110.6	1.28	6.50	.657	29.378	30.035	.861	.007785	----
156	----	603.0	66.42	9.08	1.28	50.38	51.66	1,160.7	66.28	9.10	1.29	48.94	50.23	1.43	.001232	.0012
157	----	603.0	119.2	5.06	.398	2,552.393	2,552.791	442.9	117.4	5.14	.410	2,552.209	2,552.619	.172	.000388	.000235
158	----	91.01	13.85	6.57	.671	98.758	99.429	227.4	14.66	6.21	.560	98.495	99.055	.374	.001645	----
159a	464.0	105.0	17.64	5.96	.552	95.810	96.362	300.0	17.90	5.87	.536	95.470	96.006	.356	.001187	.0012
160a	464.0	99.0	18.16	5.45	.462	95.800	96.262	300.0	17.46	5.67	.500	95.400	95.900	.362	.001207	.0012
161a	464.0	150.0	24.90	6.03	.565	96.600	97.165	300.0	25.90	5.79	.521	96.330	96.851	.314	.001047	.0012
162	----	216.1	35.93	6.02	.562	97.540	98.102	85.0	33.18	6.51	.659	97.338	97.997	.105	.001235	----
163	----	209.2	35.94	5.82	.527	97.452	97.979	85.0	34.53	6.06	.571	97.335	97.906	.073	.000859	----
164	----	52.8	12.38	4.27	.284	92.221	92.505	161.6	13.96	3.78	.222	92.172	92.394	.110	.000681	----
165	----	108.7	16.90	6.43	.643	99.879	100.522	375.0	21.57	5.04	.395	99.589	99.984	.538	.001435	----
166	600.0	564.0	114.0	4.95	.381	98.890	99.271	820.2	112.7	5.00	.389	98.617	99.006	.265	.000323	.00033
167	600.0	602.0	122.3	4.92	.376	99.332	99.708	820.2	121.35	4.96	.383	99.054	99.437	.271	.000330	.00033
168	----	295.0	48.29	6.11	.581	98.494	99.075	1,150.0	48.35	6.10	.579	97.468	98.047	1.028	.000894	----

TABLE 3.—*Elements of experiments on flumes to determine the energy slope for use in solution of flow formulas. See table 2. Difference in elements at 2 ends of reach indicates net variation from uniform flow that exists in field practice*—Continued

SHORT METAL FLUMES—FLUSH INTERIORS—Continued

Reference no.	Used in design Q'	Observed Q	Elements, upper end of reach L					Length of reach L	Elements, lower end of reach L					Retardation loss		Constructed slope s
			Area a_1	Velocity v_1	Velocity head h_1	Elevations			Area a_2	Velocity v_2	Velocity head h_2	Elevations		In reach $E_1-E_2=h_f$	Per foot $S=\frac{hf}{L}$	
						Surface Z_1	Energy gradient $E_1=h_1+Z_1$					Surface Z_2	Energy gradient $E_2=h_2+Z_2$			
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
	Sec.-ft.	*Sec.-ft.*	*Sq. ft.*	*Ft. per sec.*	*Feet*	*Feet*	*Feet*	*Feet*	*Sq. ft.*	*Ft. per sec.*	*Feet*	*Feet*	*Feet*	*Feet*	*Ft. per ft.*	*Ft. per ft.*
169	----------	98.01	26.15	3.73	0.216	97.820	98.036	288.8	24.97	3.97	0.245	97.640	97.885	0.151	0.000523	----------
170	----------	83.30	25.99	3.21	.160	97.892	98.052	288.8	24.51	3.40	.180	97.613	97.793	.259	.000896	----------
171	----------	14.69	2.54	5.78	.519	96.485	97.004	169.9	4.56	3.22	.161	96.419	96.580	.424	.002496	----------
172	----------	9.63	5.00	1.93	.058	97.591	97.649	73.5	3.91	2.46	.094	97.507	97.601	.048	.000653	----------
173	----------	102.5	33.55	3.06	.146	98.763	98.909	321.3	32.72	3.13	.153	98.624	98.777	.132	.000411	----------
SHORT METAL FLUMES—PROJECTING BANDS																
176	----------	220.79	35.49	6.22	0.602	98.813	99.415	200.0	28.50	7.75	0.934	97.960	98.894	0.521	0.002605	----------
177a	----------	42.1	----------	----------	----------	98.01	----------	140.0	----------	----------	----------	97.67	----------	----------	----------	----------
178	----------	39.2	8.40	4.67	0.339	100.000	100.339	300.5	7.11	5.51	0.472	98.840	99.312	1.027	0.003425	----------
179	----------	33.3	4.66	7.14	.793	100.000	100.793	189.0	4.11	8.10	1.020	98.985	100.005	.788	.00417	----------
180	----------	23.7	3.98	5.96	.552	50.847	51.399	635.0	4.52	5.24	.427	48.240	48.667	2.732	.00430	----------
181a	108.0	38.5	9.30	4.14	.267	98.130	98.397	590.0	10.40	3.70	.213	97.080	97.293	1.104	.001871	0.00193
182a	108.0	23.0	6.30	3.65	.207	97.600	97.807	590.0	7.45	3.09	.148	96.700	96.848	.959	.001625	.00193
183a	108.0	17.0	4.90	3.47	.187	97.330	97.517	590.0	6.50	2.62	.107	96.500	96.607	.910	.001542	.00193
184a	108.0	75.8	18.19	4.17	.270	98.980	99.250	490.0	15.34	4.94	.379	97.880	98.259	.991	.002022	.00193
SHORT CORRUGATED-METAL FLUMES																
187	----------	3.00	0.93	3.23	0.162	98.122	98.284	175.0	0.83	3.62	0.204	96.710	96.914	1.370	0.007829	----------
188	----------	16.9	6.99	2.42	.091	100.000	100.091	249.6	8.73	1.94	.059	99.780	99.839	.252	.00101	----------
189	----------	44.02	11.64	3.78	.222	93.318	93.540	73.6	11.82	3.72	.215	92.987	93.202	.338	.004592	----------

LONG WOOD-PLANK FLUMES

213	....	399.8	42.41	9.43	1.382	90.000	91.382	1,100.0	46.32	8.63	1.158	87.907	89.065	2.317	0.002106	0.002
217	....	95.5	35.75	2.60	.105	85.660	85.765	1,000.0	36.70	2.61	.106	85.420	85.526	.239	.000239	....
218	....	142.1	53.28	2.667	.111	85.780	85.891	1,000.0	54.87	2.59	.104	85.585	85.689	.202	.000202	....
219	200.0	177.5	24.44	7.26	.820	100.461	101.281	1,014.0	23.80	7.46	.865	98.420	99.285	1.996	.001969	.002
220	....	64.02	16.26	3.94	.241	88.290	88.531	616.0	19.22	3.33	.172	87.712	87.884	.647	.001050	....
221	....	64.02	18.26	3.51	.191	90.000	90.191	2,200.0	19.22	3.33	.172	87.712	87.884	2.307	.001049	....
222	....	64.02	18.26	3.51	.191	90.000	90.191	437.0	18.74	3.42	.181	89.558	89.739	.452	.001034	....
223	....	64.02	18.74	3.42	.181	89.558	.89.739	1,147.0	16.26	3.94	.241	88.290	88.531	1.208	.001053	....
224	....	16.01	12.05	1.33	.028	49.828	49.856	1,712.0	8.04	1.99	.062	49.374	49.436	.420	.000245	....
225	....	159.34	25.65	6.21	.600	90.000	90.600	400.0	25.65	6.21	.600	89.324	89.924	.676	.001690	....
226a	....	408.0	61.76	6.61	.679	49.935	50.614	1,647.9	50.50	8.08	1.015	47.510	48.525	2.089	.001268	.00085
227	....	1,104.6	222.4	4.97	.383	96.155	96.538	2,499.5	224.3	4.93	.377	95.520	95.897	.641	.000256	....
228	....	50.31	14.65	3.43	.183	90.000	90.183	195.0	13.19	3.87	.226	89.669	89.895	.288	.001479	....
229	....	50.31	13.19	3.81	.226	89.669	89.895	159.0	11.50	4.38	.298	89.289	89.587	.308	.001934	....
230	....	50.31	11.50	4.38	.298	89.289	89.587	244.0	11.63	4.32	.290	88.773	89.063	.524	.002145	....
231	....	50.31	14.65	3.43	.183	90.000	90.183	598.0	11.63	4.33	.291	88.773	89.064	1.119	.001871	....
232	450.0	419.20	58.58	7.16	.796	98.428	99.224	954.5	59.78	7.02	.766	97.384	98.150	1.074	.001125	....
233	800.0	368.81	99.12	3.72	.215	97.142	97.357	2,500.0	106.96	3.45	.185	96.030	96.215	1.142	.0004568	.0007

SHORT WOOD-PLANK FLUMES

245	....	251.8	53.71	4.69	0.341	90.000	90.341	800.0	58.90	4.27	0.284	89.795	90.079	0.262	0.000327	....
246	....	88.74	16.20	5.48	.467	84.070	84.537	500.0	19.10	4.65	.336	83.310	83.646	.891	.001782	....
247	....	153.6	17.70	8.68	1.171	84.660	85.831	700.0	23.50	6.54	.665	83.750	84.415	1.416	.002023	....
248	....	160.8	17.00	9.47	1.394	84.590	85 984	700.0	24.9	6.46	.649	83.890	84.539	1.445	.002064	....
249	....	206.9	21.00	9.85	1.508	84.990	86.498	700.0	29.20	7.08	.779	84.320	85.099	1.399	.001999	....
251	....	142.10	43.90	3.24	.163	86.320	86.483	500.0	45.15	3.15	.154	86.170	86.324	.159	.000318	....
252	....	48.16	14.53	3.31	.170	98.216	98.386	400.8	12.75	3.78	.222	97.900	98.122	.264	.000659	....
253	....	2.16	1.32	1.64	.042	90.525	90.567	606.0	1.32	1.64	.042	90.000	90.042	.525	.000868	....
258	....	92.81	37.92	2.45	.093	85.750	85.843	700.0	41.08	2.26	.079	85.590	85.669	.174	.0002486	....
259	....	65.20	21.39	3.05	.145	89.040	89.185	1,000.0	20.78	3.14	.153	88.510	88.663	.522	.000522	....
260	....	69.07	22.17	3.11	.150	89.180	89.330	738.0	22.17	3.11	.150	88.780	88.930	.400	.000542	....
261	....	43.05	37.26	1.16	.021	90.000	90.021	470.0	43.54	.99	.015	89.979	89.994	.027	.000057	....
262	....	6.97	4.25	1.64	.042	90.000	90.042	163.2	6.51	.107	.018	89.965	89.983	.059	.0003615	....
263	....	702.0	43.6	16.10	4.030	91.410	95.440	440.0	71.0	9.89	1.521	91.380	92.901	2.539	.005771	0.003
264	....	76.12	22.26	3.42	.182	124.115	124.297	1,100.0	24.64	3.09	.148	123.480	123.628	.669	.0006082	....
265	....	6.33	3.18	1.930	.058	47.450	47.508	360.0	3.76	1.634	.041	47.229	47.270	.238	.000661	....
266	....	19.35	11.47	1.69	.039	90.000	90.039	1,014.4	10.71	1.81	. 051	89.664	89.715	.324	.000319	....
267	....	21.96	3.24	6.78	.714	90.000	90.714	400.0	4.23	5.19	.419	86.019	86.438	4.276	.010690	....
268	....	398.0	60.1	6.63	.663	97.845	98.528	650.0	49.8	8.00	.995	96.290	97.285	1.243	.001912	....
269a	....	435.0	100.8	4.32	.290	51.050	51.340	513.0	101.0	4.31	.289	50.820	51.109	.231	.000450	.0003
271	....	23.64	4.48	5.28	.433	90.000	90.433	746.3	6.36	3.72	.215	87.303	87.518	2.915	.003906	....
272	....	67.25	53.82	1.25	.024	49.075	49.099	500.0	51.49	1.30	.026	49.033	49.059	.040	.000080	....
274	....	15.72	12.45	1.26	.025	90.000	90.025	672.1	10.68	1.47	.034	89.568	89.602	.423	.000629	....

TABLE 3.—*Elements of experiments on flumes to determine the energy slope for use in solution of flow formulas. See table 2. Difference in elements at 2 ends of reach indicates net variation from uniform flow that exists in field practice*—Continued

LONG WOOD-STAVE FLUMES

Reference no.	Used in design Q'	Observed Q	Elements, upper end of reach L					Length of reach L	Elements, lower end of reach L					Retardation loss		Constructed slope s
			Area a_1	Velocity v_1	Velocity head h_1	Elevations: Surface Z_1	Elevations: Energy gradient $E_1=h_1+Z_1$		Area a_2	Velocity v_2	Velocity head h_2	Elevations: Surface Z_2	Elevations: Energy gradient $E_2=h_2+Z_2$	In reach $E_1-E_2=h_f$	Per foot $S=\frac{h_f}{L}$	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
	Sec.-ft.	*Sec.-ft.*	*Sq. ft.*	*Ft. per sec.*	*Feet*	*Feet*	*Feet*	*Feet*	*Sq. ft.*	*Ft. per sec.*	*Feet*	*Feet*	*Feet*	*Feet*	*Ft. per ft.*	*Ft. per ft.*
300																0.001
301	656.0	429.0	43.9	9.77	1.48	102.59	104.07	2650.0	46.01	9.32	1.35	97.42	98.77	5.30	0.002000	.002
302	656.0	429.0	38.9	11.03	1.88	100.42	102.30	1200.0	40.00	10.73	1.79	98.08	99.87	2.43	.002025	.002
303	656.0	417.5	45.49	9.18	1.31	102.74	104.05	2650.0	52.05	8.02	1.00	97.82	98.82	5.23	.001974	.002
304	656.0	440.0	45.36	9.70	1.46	102.77	104.23	2650.0	53.79	8.18	1.04	97.94	98.98	5.25	.001981	.002
305	656.0	383.8	37.8	10.15	1.60	102.09	103.69	2650.0	48.6	7.90	.97	97.69	98.66	5.03	.001898	.002
306		246.6	48.88	5.05	.396	401.779	402.175	2680.0	48.39	5.10	.404	400.516	400.920	1.255	.000468	.0004
307		55.41	15.21	3.64	.206	100.987	101.193	1500.0	13.98	3.96	.244	100.000	100.244	.949	.000633	
308		65.87	16.61	3.97	.244	98.906	99.150	1000.0	16.78	3.93	.239	98.371	98.610	.540	.000540	

SHORT WOOD-STAVE FLUMES

Reference no.	Used in design Q'	Observed Q	Area a_1	Velocity v_1	Velocity head h_1	Surface Z_1	Energy gradient E_1	Length of reach L	Area a_2	Velocity v_2	Velocity head h_2	Surface Z_2	Energy gradient E_2	In reach h_f	Per foot S	Constructed slope s
320		48.16	12.41	3.88	0.234	98.474	98.708	297.0	12.23	3.94	0.241	98.310	98.551	0.157	0.000529	
321		182.9	29.24	6.25	.607	93.542	94.149	368.3	32.27	5.67	.500	93.396	93.896	.253	.000687	
322	230.0	193.6	25.66	7.55	.884	98.255	99.139	444.1	28.75	6.74	.705	97.937	98.642	.497	.001119	0.0014
323		2.20	0.94	2.33	.084	100.455	100.539	905.0	1.88	1.17	.021	100.000	100.021	.518	.000572	
324		112.5	20.71	5.43	.458	100.000	100.458	353.0	20.30	5.54	.477	99.686	100.163	.295	.000836	
325		112.5	20.30	5.54	.477	99.686	100.163	97.8	20.00	5.63	.491	99.576	100.067	.096	.000982	
326		23.12	7.14	3.24	.163	97.303	97.466	313.0	6.79	3.40	.180	97.023	97.203	.263	.000840	

FLUME CHUTES: COMPUTED FOR MEASURED CROSS SECTIONS AND MEASURED VELOCITIES; METAL CHUTES

440	----------	61.0	2.50	27.82	12.02	97.275	109.293	1500.0	2.55	27.27	11.561	11.885	23.446	85.85	0.05723	----------
441	----------	26.5	1.56	19.37	5.830	97.010	102.840	1500.0	1.70	17.77	4.910	11.640	16.550	86.29	.05753	----------
442	----------	59.2	7.22	--------	--------	95.54	----------	370.3	7.10	----------	--------	83.60	----------	11.91	.032164	----------
443	----------	151.4	12.46	--------	--------	96.29	----------	370.3	10.30	----------	--------	84.17	----------	11.06	.035991	----------

FLUME CHUTES: COMPUTED FOR MEASURED CROSS SECTIONS AND MEASURED VELOCITIES; WOOD PLANK CHUTES

450	----------	1.24	0.13	9.76	1.481	91.540	93.021	598.2	0.086	14.45	3.245	58.440	61.685	31.34	0.05238	----------

CHUTES COMPUTED FOR MEASURED SECTIONS BUT WITH VELOCITY, V, DETERMINED AS Q/A; METAL CHUTES

440′	----------	61.0	2.50	24.40	9.245	97.275	106.520	1,500.0	2.55	23.92	8.893	11.885	20.778	85.74	0.057163	----------
441′	----------	26.5	1.56	16.99	4.490	97.010	101.500	1,500.0	1.70	15.59	3.780	11.640	15.420	86.08	.057387	----------
442′	----------	59.2	7.22	8.20	1.05	95.54	96.59	370.3	7.10	8.33	1.08	83.60	84.68	11.91	.032164	----------
443′	----------	151.4	12.46	12.15	2.30	96.29	98.59	370.3	10.30	14.70	3.36	84.17	87.53	11.06	.035991	----------

CHUTES COMPUTED FOR MEASURED SECTIONS BUT WITH VELOCITY, V, DETERMINED AS Q/A; WOOD-PLANK CHUTES

451′	----------	6.16	0.76	8.09	1.017	92.895	93.912	516.0	0.063	9.82	1.499	78.615	80.114	12.80	0.02480	----------

DEDUCTIONS FROM CAPACITY TESTS

The primary purpose of the experiments was to secure the values of friction factors for a wide range of conditions in flume flow. Results of this part of the study are given in the values of Kutter's *n* recommended for various conditions. (See p. 51.)

The lack of uniform flow, even under conditions that appeared to warrant it quite fully, indicated that complete measurements, including the areas at the ends of the reach under consideration, must be obtained in order to compute the proper effective slope—that of the energy gradient.

The friction factors are higher than were anticipated. Effects of algae and insect growth were the rule rather than the exception, especially near the upper ends of long flumes. They were much greater than any effects of age. The bottoms of concrete flumes were almost invariably much rougher than the sides. This is partly due to methods of construction and partly to erosion by detritus. Metal flumes have been improved greatly in ability to maintain capacity. The older metal flumes were usually out of line at every sheet joint. The joints used at present do not have this defect. On the other hand, paint was seldom used on the early metal flumes and is standard practice now. The unpainted sheet was smoother than the painted one, as a rule. Paint and enamel coats should be free from long ridges, due to flowing of excess paint. The mosaic effect called "alligatoring" is more noticeable for some paints than others. (See pl. 2, B.) Likewise, the early flumes were often deficient in number of carrying rods. This resulted in scalloping of the sheets, causing a noticeable retarding effect. At present sufficient rods are used and the flume sheets present a smooth alinement.

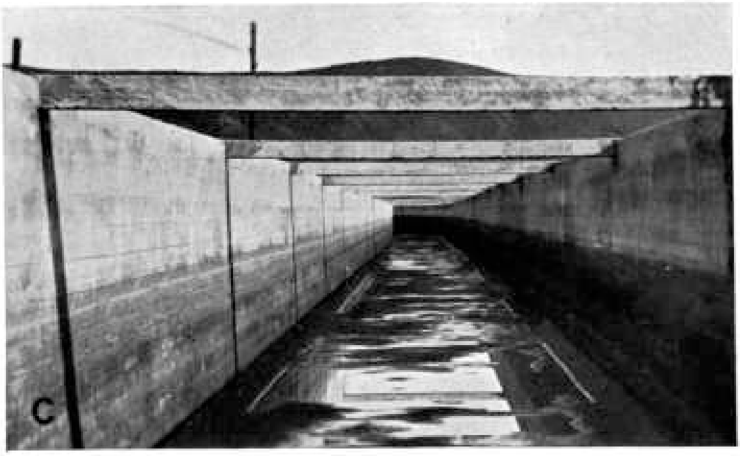
Wood-plank flumes show greater changes by age alone than metal or concrete flumes. Battens should be avoided as they collect silt and moss deposits that have great retarding effect. Either plank or stave flumes are subject to raveling of the bottom by rough abrasives. Such erosive action smooths a metal flume, but wears it out.

FLOW CLASSIFICATION IN FLUMES

Before using the formulas necessary to the computations of flume flow, it is advisable to study the conditions under which a flume is merely an open channel at high velocity and the conditions that develop the more usual nonuniform flow in a uniform channel.

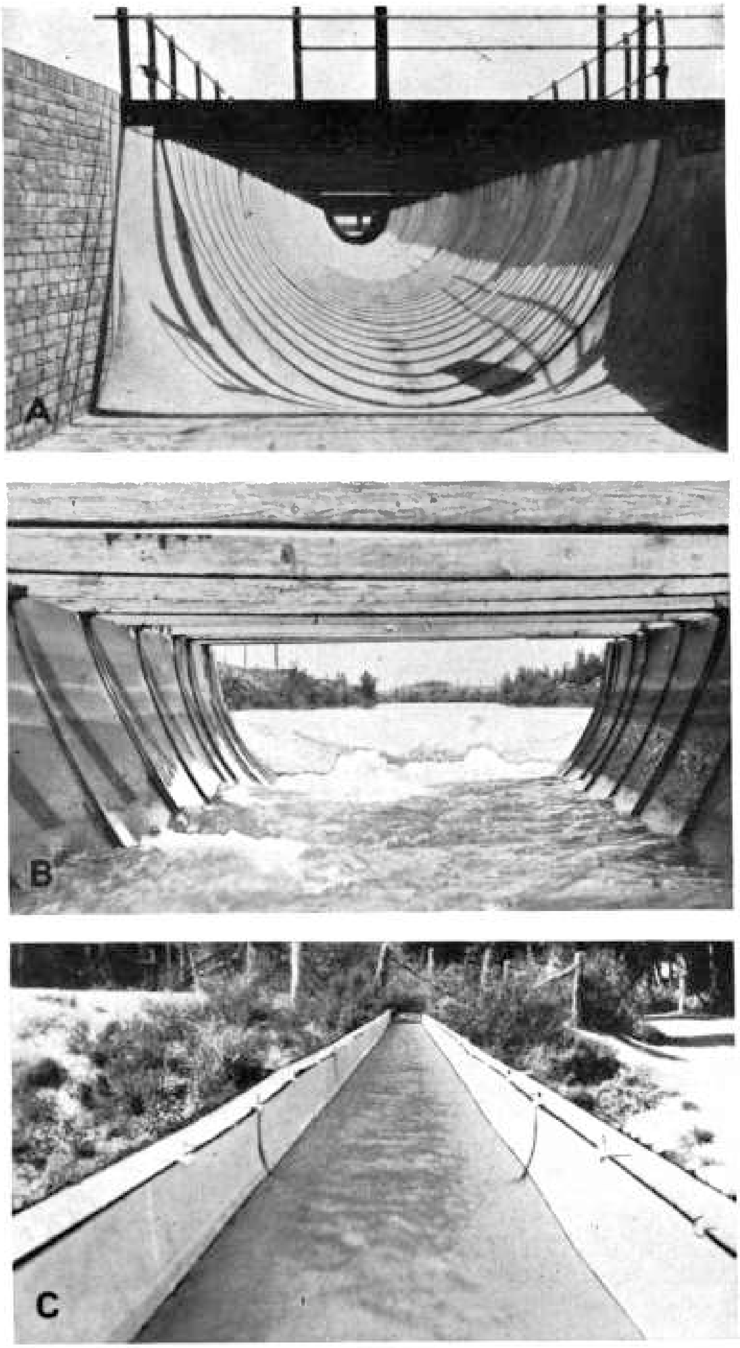
DESIGNED FLOW AND ATTAINED FLOW

Both long and short flumes are usually designed for the idealized condition of uniform flow at normal depth, d_n, throughout their lengths, using Kutter's or other flow formula to develop the dimensions of the elements within the flume. The "paper" water prism is then set in the scheme of levels with due provision for a drop in the water surface at the inlet sufficient to increase the velocity from canal to flume and with a corresponding provision at the outlet for a reasonable recovery of velocity head as the velocity is decreased from flume to canal. Such uniform flow and velocity-head changes in water surface are indicated in figure 2, A. Sometimes construction cost can be reduced by designing for a drop-down curve as hereinafter described.



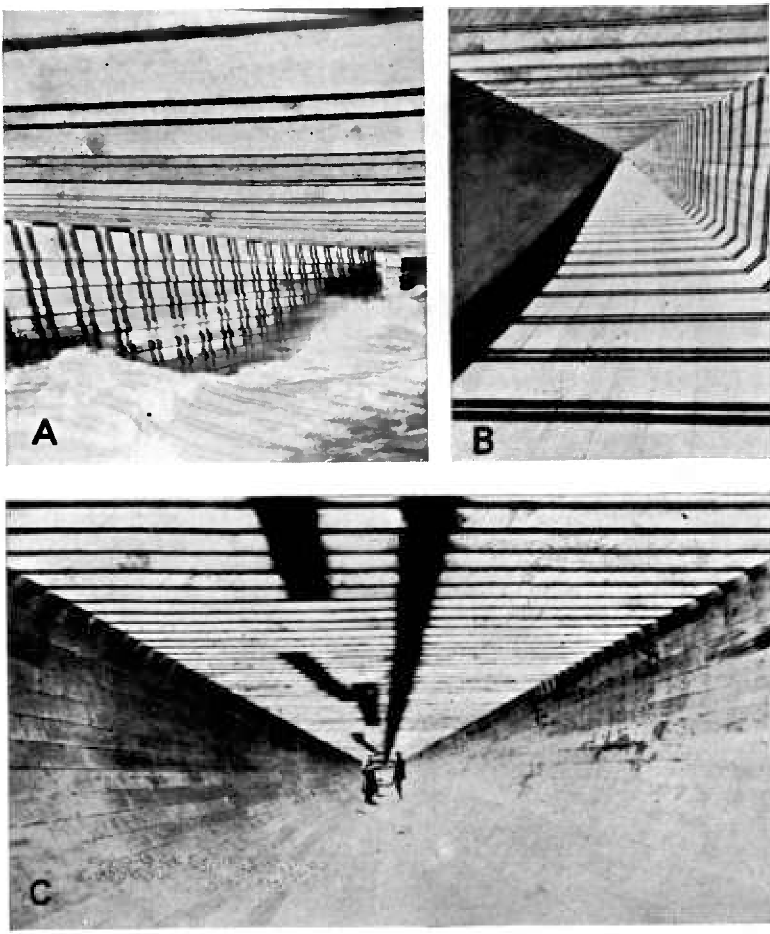
TYPICAL INTERIORS: CONCRETE.

A, Sulphur Creek wasteway, Yakima project, Washington, nos. 424 to 430, inclusive, cast in wood forms; backfilled. B, Main canal, King Hill project, Idaho, no. 14, precast side units, joined at pilasters, with forms still in place. C, High-line canal, Lindsay-Strathmore Irrigation District, Calif., no. 16, gunite concrete shot from the outside against smooth board side forms. Rough bottom, typical of gunite shot directly and not given smoothing treatment.



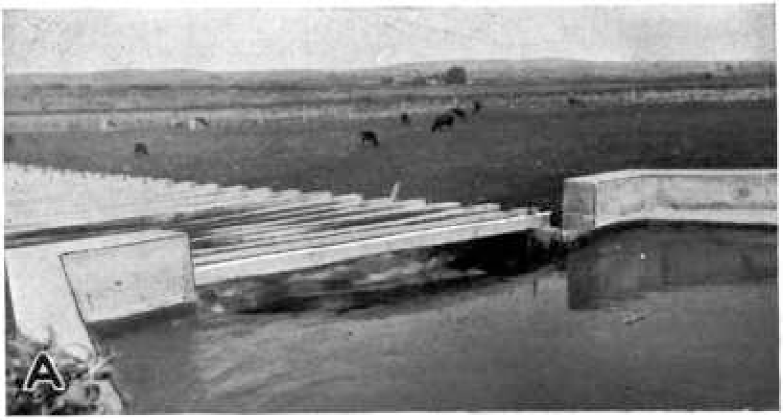
TYPICAL INTERIORS: METAL.

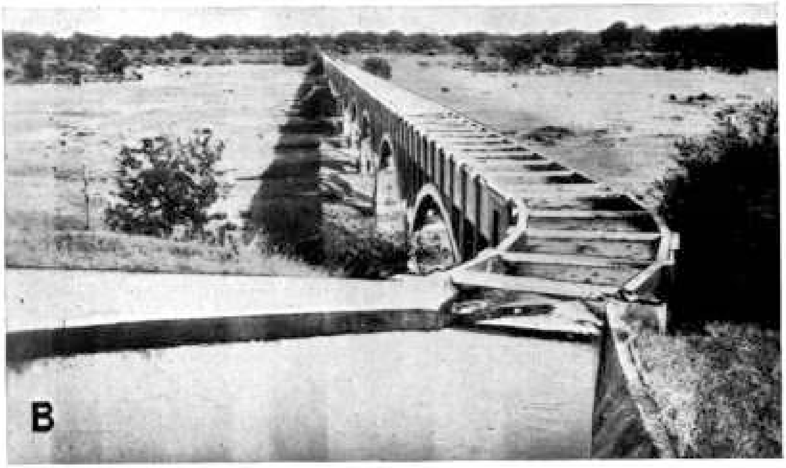
A, Johi flume, India, a short structure with flush interior (L type, see fig. 1 H, *d*). Note padded transition between rectangular inlet and metal flume. B, Tule-Baxter Irrigation District, Calif., looking upstream into inlet. Like no. 112. Note projecting bands characteristic of G type of metal flume and "alligatoring" typical of some paints. (See fig. 1 H, *b*.) C, E. C. Hauser flume, near Sacramento, Calif., no. 155. A small flume of longitudinal sheets and with flush interior of L type. (See fig. 1 H, *d*.)

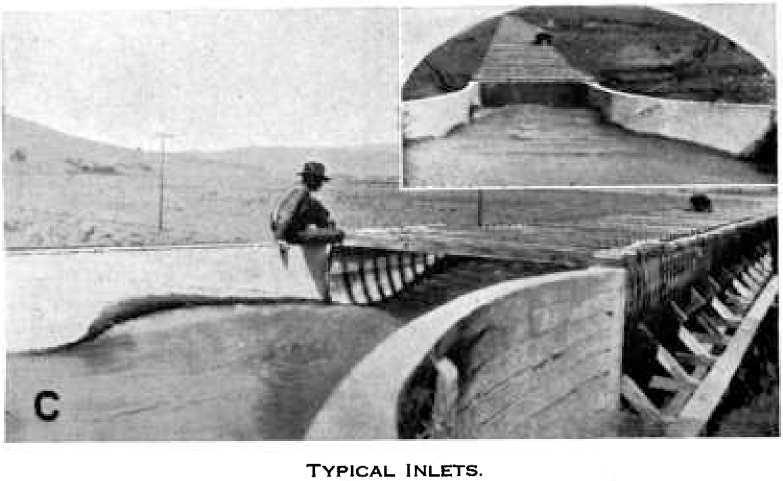


TYPICAL INTERIORS: WOOD.

A, Old type plank flume with battened horizontal joints. (In some flumes, vertical cracks also are battened.) Note waves 3 to 4 feet high in this by-pass flume for construction of dam. High velocities and sharp angular bends developed rough water. B, Olmsted flume, near Provo, Utah, no. 232. Modern planked structure with splined joints. (See fig. 1.) The fillet in lower corner is formed by beveling heavy squared timber, grooved to take side and bottom planks; the whole side structure is snugged together with long bolts from sill to cap. C, Central Oregon Canal flume, near Bend, Oreg., no. 301–305. Of continuous construction, creosoted Douglas fir staves, 12 feet in diameter. Mid-diameter is 1.2 feet below bottom of caps, and computations are based on full half-circle. Freeboard projects inward, making good wave deflector. (This flume used for examples and characteristic curves, pp. 65–89.)





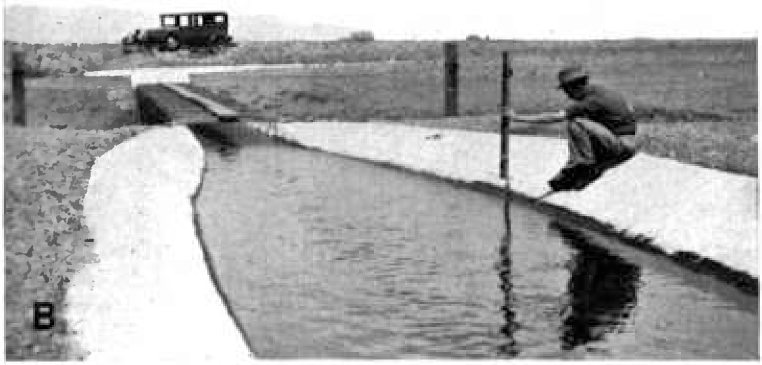


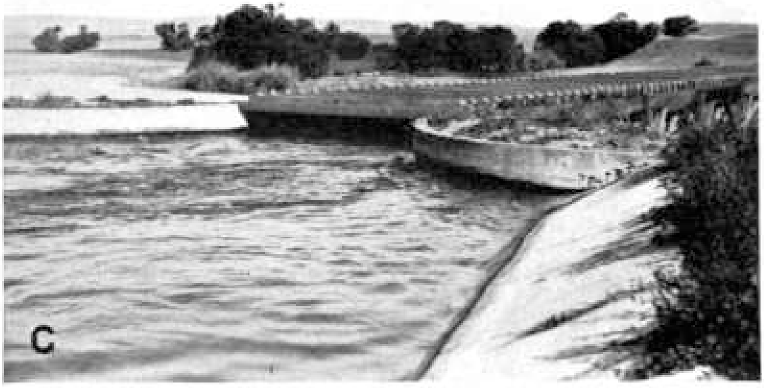
TYPICAL INLETS.

A, "Square" inlet, Mora flume, near Caldwell, Idaho, no. 176. B, Flared-wing inlet, Cottonwood Creek flume, California, no. 71. Note warp of transition from canal side slopes to vertical at flume entrance. C, Warped inlet, McEachren flume, King Hill project, Idaho, no. 322. Even carefully designed inlets may develop rough water for discharge well below design Q.

TYPICAL CYLINDER-QUADRANT-WALL INLETS.

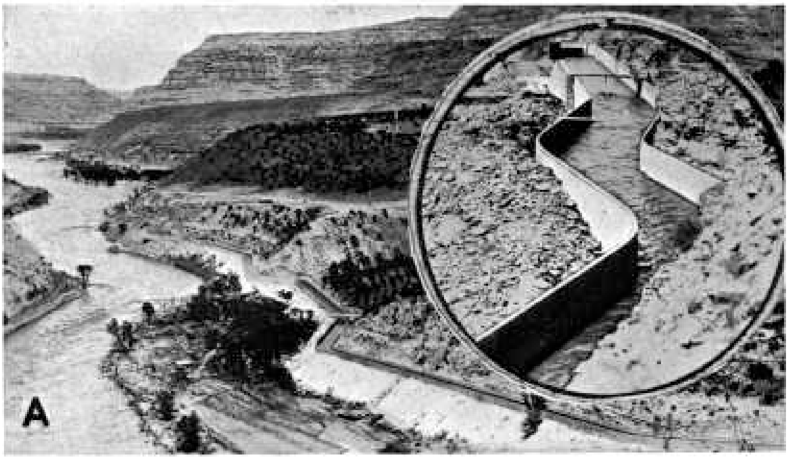
A, E. C. Hauser flume, near Sacramento, Calif., no. 155. Inlet from large earth ditch into small metal flume. (See pl. 2, C.) B, Standard inlet structure of West Side Stanislaus Irrigation District, Calif. Like no. 127. Note tangent distance between cylinder wall and metal flume for development of surface drop at full capacity. C, A short rectangular concrete flume on Borel Canal, Calif. Note the perfect tangential line of contact between wing and flume body. Water-stage recorder kept in small house at right.





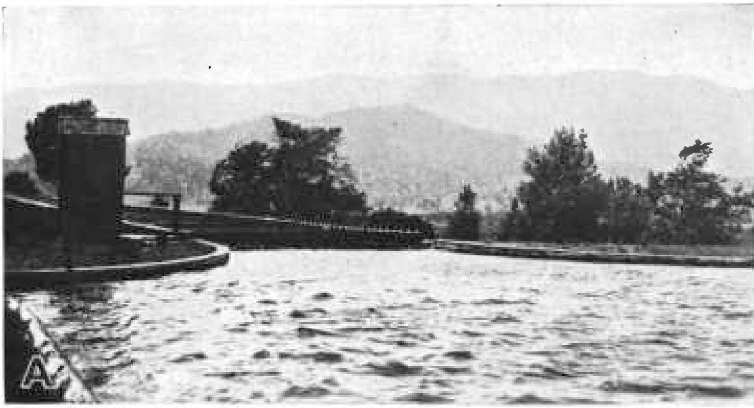
TYPICAL OUTLETS.

A, Main flume, Central Oregon Canal, near Bend, no. 301; warped flare. B, Standard outlet on West Side Stanislaus Irrigation District, Calif. C, Quadrant-wall outlet is not recommended. While very successful as an inlet, this form of outlet is not much improvement over the 45° wing wall.



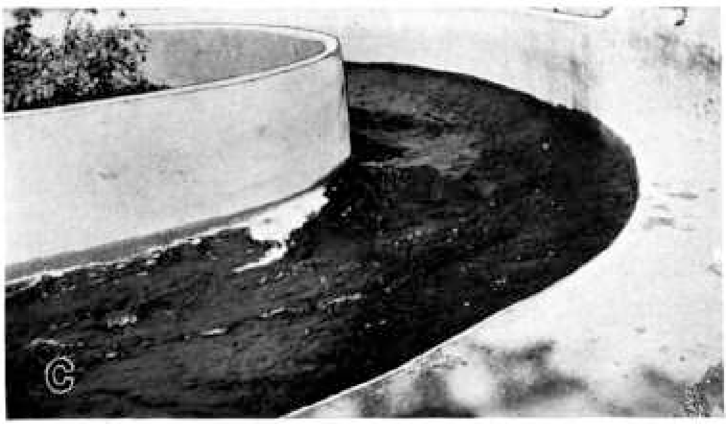
TYPICAL ALINEMENT, LONG FLUMES.

A, Orchard Mesa flume, Grand Junction, Colo., no. 18. Inset shows detail at far end. Note rounded hends rather than curves. B, Tieton flume, Yakima project, Washington, no. 10. As originally constructed this structure was called a "lining." After reconstruction, if not before, this is a hench flume. C, Bowman-Spaulding Conduit, Nevada Irrigation District, Calif., nos. 105–109; a metal flume on trestles in rugged glaciated canyon with steep hard-rock sides.



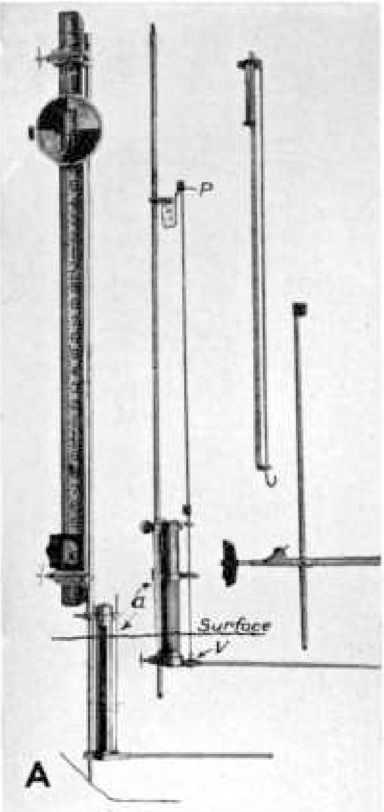
MAXIMUM CAPACITY.

A, Erskine Creek flume, Borel Canal, Calif., no. 167. Cylinder-quadrant inlet and good adjustment of relative levels permit operation with a freeboard throughout flume of but 5 inches. B, Inlet end, Brooks Aqueduct, Canadian Pacific Railway irrigation project, Alberta, Canada, nos. 23 to 38, inclusive. Two-foot freeboard at outlet. C, Agua Fria flume, near Phoenix, Ariz., no. 104. Drop-down curve develops 2.46-foot freeboard at outlet. Views B and C show that capacity of long flumes cannot be increased by lowering the stage at the outlet. For first 3,000 feet freeboard was quite constant at 1 foot.

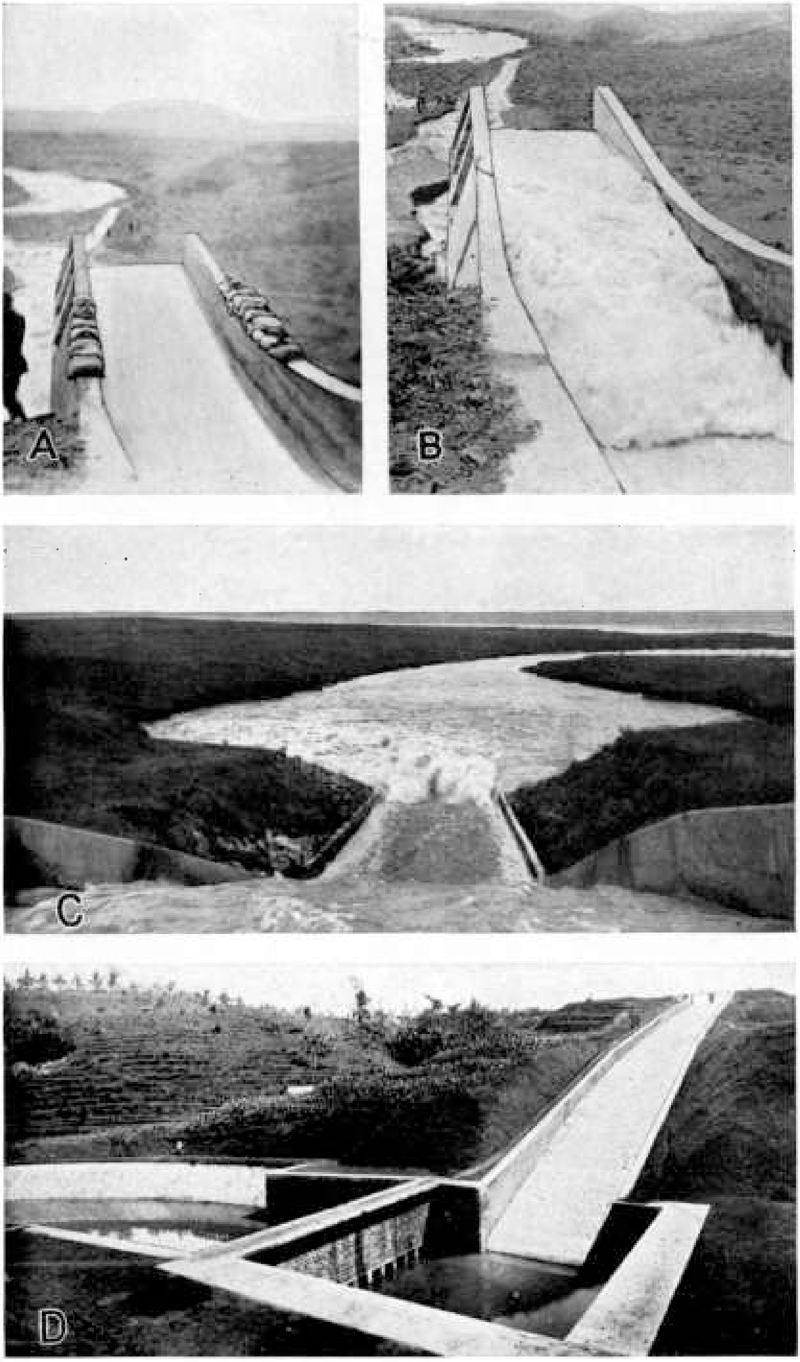


TYPICAL SHARP CURVATURE.

A, Tahoe flume, Drum Canal, Calif., no. 72. Note curves are made with straight forms. B, Tieton flume, Yakima project, Washington, no. 10. (Fig. 1 C.) Looking in direction of flow around sharp curve. Like C below, water rises on outside of curve. Upward current near surface at outside of curves is contrary to ideas in many technical articles. C, Cross-over flume, California, no. 15. Note smooth fountain head rising on outside of curve and rolling toward center.



A, Precision gages, used to determine exact average distance of water surface down from known elevation (P) or up from any point on the bottom. Hoff current meter used to determine Q. B, Sinuous alinement on Tiger Creek Conduit, Calif. View shows 430° of curvature in a length of some 780 feet. Excessive curvature feasible when necessary. All curves shown except first one are of 75-foot radius, the minimum used for the conduit. C, Flow over a brink at critical depth. Picacho flume, Rio Grande project, N.Mex. Drop-down curve begins far upstream at normal depth.



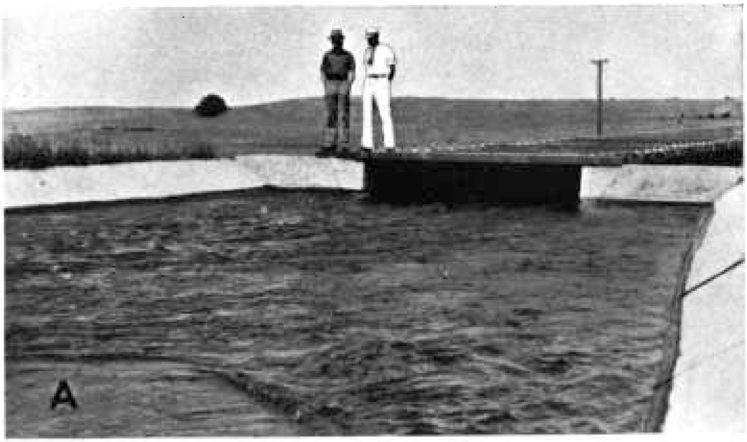
TYPICAL CHUTE FLUMES.

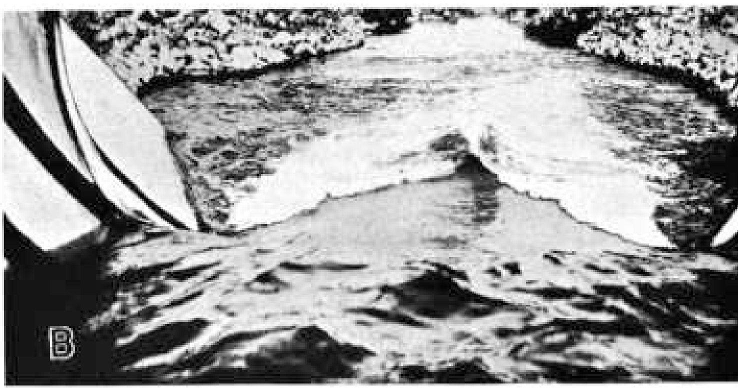
A, Water flowing into Lahontan Reservoir, Nevada, comes down a steep incline behind camera and shoots over up-turned end of the chute. B, Same as A, above, with lesser flow showing jump which occurs for all flows less than about 90 second-feet. C, Water feeding Lake Newell on Canadian Pacific Railway project in Alberta, Canada. Energy dissipated in jump. D, A chute from high-level to low-level canal in Java. Entering the pool at the bottom, much of the energy will be dissipated in the resulting jump.

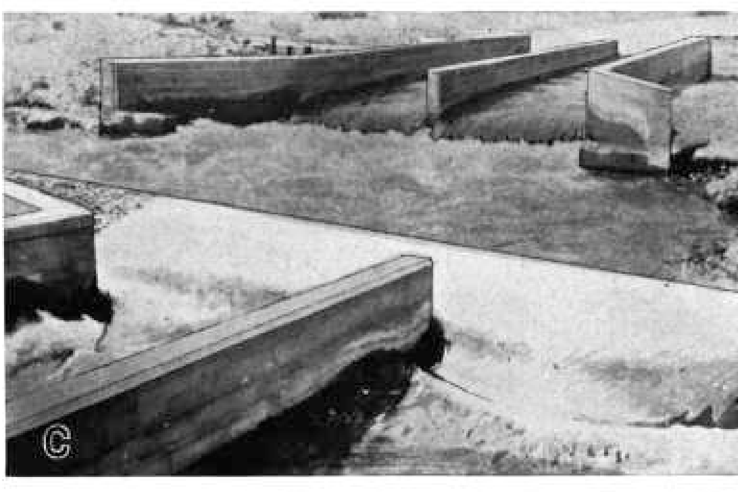
TYPICAL FLUME FLOWS.

A, Approximately uniform flow in a long flume. No. 15. Making current-meter measurement in a rectangular flume of troweled gunite. B, Flow in Dalroy chute, Alberta, Canada, no. 443. Concrete taper section used at inlet, while flow is accelerated to normal. Metai flume with projecting compression bands causes excessively turbulent flow. C, Complex flow in a riveted sheet-metal flume. Excess drop for the amount of flow caused critical flow over inlet brink. The velocity is then faster than critical for many feet of flume. Mild slope insufficient to maintain shooting stage so jump results.

A, Central Oregon Irrigation District main canal, near Bend, Oreg., nos. 301 to 305, inclusive. A true semicircular flume 12 feet in diameter. Test reach began at lower end of tangent in foreground and ended at outlet beyond last curve in distance. B, Tiger Creek Conduit on Mokelumne River, Calif., Pacific Gas & Electric Co. Inset shows flow of 537 second-feet. No. 39. Carriage hanging on wall used as traveling platform so engineer can secure precision measurements down to water surface from elevation spots on inside edges of flume.







A, A "square" outlet, Le Grande flume, Merced Irrigation District, Calif., no. 168. Note backlash eddy typical of all outlets except long tapers. B, Depressed outlet, with angled wings. Note dark, smooth characteristics of critical flow near hrink in foreground. C, Outlet and inlet of a short, raised flume in San Luis Valley, Colo. Canal crosses wide drainage channel, shown running full head of water. Ahrupt entry causes sudden generation of velocity in flume, with consequent cavitation shown by the contraction of the jet at sides of channel in lower view.

A, Typical water surface condition in Tiger Creek flume. Note smooth rising surface on outside and choppy surface on inside of curves. B, Excess fall at the inlet end of a long flume of mild slope will not insure a shooting flow the full length of a flume. C, The simple tapered inlet meets the flume at angles that develop "cavitation" and surface plunges that reduce the ultimate maximum capacity. Note typical accretions of silt coating which cracks and wrinkles when dry and still remains rough when wet again.

A, Happy Canyon flume, Uncompahgre project, U.S.B.R., Colorado, nos. 145–150. Note tongue to divide water equally between two barrels of twin flume. B, No. 2 flume, lateral 2, Columbia Irrigation District, Washington. Creosoted stave flume (no. 320), then a reinforced concrete inverted siphon under railroad tracks, then a wood plank flume (no. 252).

Conditions attained in the field are generally quite different from those assumed in design. This fact causes uniform flow to be approximated only in long, straight flumes. In short flumes even an approximation to uniform flow takes place only as a coincidence. The field tests listed in tables 2 and 3 certainly express a cross section of field practice over a wide scope, yet uniform flow was seldom indicated by precise measurements. Table 3 shows the extent of divergence in the net tapered flow found by comparing the elements at the upper and lower ends of the reach under consideration. This divergence in flow is so general and so marked that it is given space in order to bring out the energy-slope factor finally used in the computations of the friction factors. Obviously, use of the slope of the channel bed, or of the water surface, would have yielded an incorrect result. Usually, the conditions that develop the tapering flow have been known in the rough but have not been evaluated so as to be given detailed consideration in design. In general these conditions may be listed as follows:

(1) Lack of conformity between design value of n and attained value, especially if the influence is not evenly distributed along the flume surface. Algæ, insect life, curvature, lime accretion, sand, and gravel are among the items that may give values of n quite different from that assumed.

(2) If designed flow is close to critical flow, slight lack of conformity in the values of n may throw the flow to the alternate depth stage. If conditions now vary in the flume, the stage may pass back and forth between the two stages of flow, either with or without the jump.

(3) A water stage in the canal below any flume flowing at streaming velocity, different from that assumed, will cause a drop-down curve or a backwater curve. In a long flume this will gradually taper back upstream until uniform flow is again approximated. In a short flume such influence will extend throughout the length.

With this conception of a lack in conformity between design and attainment, the main classes of flow that can be developed under given conditions can be listed. The formulas and examples given later furnish methods of bringing these conditions into a given problem and evaluating them in design so that the difference between design and attained figures may not be of moment.

FLUME TYPE A-C

A long flume with normal velocity below the critical (p. 48).

Flow class A-1. Normal depth greater than critical ($d_n > d_c$); flume long enough to develop approximately normal flow. The ordinary solution of flow formulas for any given three of the four elements, S, V, R, and n, gives the fourth element for normal flow. For a long structure, more or less sinuous, the flow approximates the values of the elements as assumed, varying slightly both above and below the design figures even though they were correctly chosen for the uniform flow assumed. Curvature, incidental gate and waste structures, inverted-siphon canyon crossings, etc., tend to induce minor modifications from the true normal.

Taper-flow reaches become dominant in short flumes. These will be discussed later. The conditions involved are also applicable to the extreme ends of long flumes.

FLUME TYPE B-C

A short flume with nonuniform flow and normal velocity below the critical ($d_n > d_c$).

While a short flume is usually designed as to the water prism in the flume proper in the same way as for a long flume, with uniform flow, this requires complete conformity of so many elements that it is attained only as a coincidence.

Flow class B-1. Normal velocity slower than critical and actual velocity slower than normal; ($d_n > d_c$ but $d > d_n$).

The depth will be greater than normal when the tail-water stage is checked up by a foul canal or a throttled structure until it is higher than $k + d_n + h_r$ at the flume outlet. This will raise the surface throughout the flume up to the inlet. It will take the form of the backwater curve and the solution can be approximated as in example shown on page 77.

Flow class B-2. Normal velocity slower than critical, but actual velocity between normal and critical; that is, $d_n > d_c$ but d is between d_n and d_c.

The depth will be less than normal when the tail-water stage is below $k + d_n + h_r$. This will speed up the water toward the flume outlet and form a drop-down curve throughout the length of the short flume. The limit of this condition takes place when tail-water stage equals $k + d_c + h_r$. At this time water is just at critical depth at the outlet brink and further lowering of the tail water will not increase the flow nor change the drop-down curve. The latter can be traced by methods given in example on page 78. As this is a short flume, the drop-down curve will still be in effect by the time it has been traced back to the inlet. That is, d will still be less than d_n, which means that there is excess drop available between the canal above and the stage in the flume proper just below the inlet. If this excess is moderate, the steeper energy gradient will generate a faster velocity in the canal above in the form of another drop-down curve. Again the limit of this curve in the canal above will come when a control at critical depth is developed on the brink at the inlet transition.

If there is still excess drop, water will enter the flume at a velocity faster than critical and the next flow class is developed.

Flow class B-3. A flume with flow at the upper end faster than critical but normal flow slower than critical; $d < d_c$ but $d_n > d_c$.

Flow immediately below the inlet control at d_c is almost unavoidably faster than the critical. However, the premise is that the slope of the flume is not sufficient to maintain velocity faster than the critical so the hydraulic jump will probably occur somewhere in the upper reaches of the flume with the drop-down curve on the surface from that point to the outlet. In many flumes, flows at but partial capacity develop this control at the inlet transition and result in a jump somewhere in the upper reaches of the flume (pl. 12, C). The flow beyond the jump may be uniform, drop-down or backwater, depending on the stage of tail water in regard to outlet depth and recovery of velocity head, as outlined above.

FLUME TYPE A-D

Long flumes at a velocity faster than critical.

Such a flume is usually a "chute" or inclined drop structure with very high velocities. However, this type of flow may occur in a flume not intended as a chute. Such was the case in the flume used for the example (p. 67).

Flow class D-1. A long flume with normal velocity faster than critical; that is $d_n < d_c$ and d approximating d_n beyond the initial reaches of the flume.

Even though a flume is designed for streaming flow a little slower than the critical, the attained value of n, at first, may be much lower than the design n and the resulting normal velocity for the new flume will be faster than the critical and consequently the depth will be less than the critical depth (p. 17).

A flume chute usually has water delivered to it from some relatively low-velocity canal. For the velocity to increase from the streaming stage to the fast, shooting stage it must go through critical depth. This usually occurs at the first brink where the grade breaks sharply downward. Water delivered to a chute from a gate structure is usually at shooting stage when it emerges from the orifice. Immediately after the critical-flow stage is reached the water is at shooting stage but is still slower than normal, that is $d < d_c$ but $> d_n$. Hence, the flow will be quickly accelerated until d_n is approximated. This acceleration will take place as head of elevation is made available in excess of that necessary to overcome friction.

RECOMMENDED VALUES OF RETARDATION COEFFICIENTS

The following discussion will be in terms of Kutter's n. However, table 2 shows that Manning's n' conforms quite closely to Kutter's n throughout the range of values usually found in flumes, say from 0.011 to 0.016 or slightly more.

In planning for a complete new conduit there may be considerable elasticity in the selection of the various hydraulic elements. For a replacement structure, however, the total fall available may be practically nonvariable and all other elements must conform to that. The engineer is required to exercise his judgment as to the value of n in several ways, illustrated in the following two groups: [12]

The design of a flume to be erected in the future:

(1) The value of n to be substituted in a flow formula to determine the approximate mean velocity, V, hence the flow, Q, for an assumed long flume of definite material, dimensions, shape, and slope. The usual design set-up is for full capacity, with a definite freeboard at normal flow. Several trials may be necessary before the resulting Q approximates the design quantity desired. Sometimes where the slope is elastic, a section area and velocity may be assumed that must yield the desired flow, Q, and the slope be then computed that will give the assumed velocity for the value of Kutter's n determined upon as best fitting operation conditions at the flume location.

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[12] Before selecting a value of n, the engineer should review the comments regarding long and short flumes under Flow Classification. A more conservative value of n should be used for long flumes since there is less opportunity for changing the effective slope in case too low a value of n is selected.

(2) A similar study of a "paper" flume, to be equated against competing structures such as an inverted siphon pipe of concrete, steel or wood, or different flume types or materials. A bench flume will often be equated against a tunnel for various portions of its length, or against an open canal, lined throughout. Occasionally a combination of these is studied; as, for example, a flume side or a flume side-and-bottom construction with a rock cut or a lined bank on the upper side. For studies of this nature, the proper relative friction factors for all the conduit types are probably more important than the specific factor that must be applied in case (1), above. Of course very conservative factors should not be selected for one type or material of construction and very lenient factors for another.

Studies of a flume already in service:

(1) The determination of the computed maximum capacity of a flume for which a capacity flow has not as yet been available, or has not been measured.

(2) The determination of the feasible increase in capacity by improvement of part or all of the containing surface. This improvement may be by permanent change in surface, or by temporary, periodic, or permanent removal of the cause of capacity reduction.

(3) The determination of a feasible increase in capacity by (*a*) improvement of inlet or outlet structures, (*b*) lowering the stage of the tail water to secure a "good get-away", as irrigators sometimes express it, (*c*) the lowering or raising of the whole structure where the flume is very short.

(4) The determination of the location and disposition of flashboards or side posts along a wood-plank flume to retard flows less than the maximum in order to keep all the side walls reasonably wet and thus prevent development of cracks and consequent loss of water. Unless otherwise stated, a velocity less than critical is assumed.

In the selection of a value of n for use in computations, it should be remembered that all experimental data on capacity of both open and closed conduits show well-authenticated cases of extremely favorable friction factors. The values attained in practice are higher than should be anticipated in design. Available experimental data were used in preparing the following recommendations as to various values of Kutter's n:

$n=0.009$. Most of the older works on hydraulics head the list of values of n for various materials with this value "for well planed timber." This description fits perfectly the new flume of surfaced lumber or of machined-wood staves without chemical treatment, but experience indicates that values given farther down the list should always be used in design. Experimental data available to Kutter (*10, p. 163*) showed values from 0.0084 to 0.0097 for two flumes, one of which was about 4 inches and the other about 7 inches wide. The lower values of n were found for flows faster than the critical. These flumes were but one board wide and one deep, without cracks or battens. These data obtained in very small flumes, are entirely too meager to be used as a basis for the use of "0.009 for well-planed lumber" in general design for commercial structures. Experience dictates a minimum value some 0.004 higher; i.e., $n=0.013$ for commercial design.

$n=0.010$. In the old list attributed to Kutter this value is suggested "for neat cement." It, too, is based on insufficient data—2 experimental channels, 1 rectangular and 1 semicircular. No field use of coatings of neat cement is known to the writer. This value may be taken as the lowest one attained in field tests where conditions approaching the ideal are maintained. It is too low to be anticipated in the design of any flume with velocities below the critical. For flows much faster than the critical, in chute flumes, this value may be used in the computation of velocity and its related trajectory of the jet as water reaches the end of the chute. (See chute flumes, p. 89.)

$n=0.011$. In the original list attributed to Kutter this value is suggested "for cement with one-third sand." This description applies more closely to our present idea of Portland-cement concrete than any other description in the old list, especially for mortar coats generally applied as a mixture of cement and sand in equal parts. The above value is often attained in very smooth circular pipes operating as flow lines; that is, with a free-water surface, a condition which is the equivalent of that prevailing in a covered circular flume. This value might be used in design in exceptional cases, where the water is free from algæ and larval growths and the cover will minimize moss growth. It has also often been used in the designing of metal flumes without projecting interior bands. Early experimental data indicated this value for long straight flumes under the best of conditions. However, age generally has affected materially such flumes, as the single carrier rod formerly in use (fig. 1, H*c*) was not sufficient to prevent opening of sheet joints and resulting offsets of sheet edges, thus reducing the carrying capacity.

Many old metal flumes designed with a value of $n=0.011$ have given little or no trouble with respect to capacity, probably for the reason that they were relatively short and a small amount of heading-up in the canal above the flume was sufficient to develop the requisite additional slope of the energy gradient. (See fig. 2, B.)

The present understanding of the use of $n=0.011$ for flume flow may be summed up as follows: It will sometimes be attained in new unpainted metal flume of types *c*, *d*, and *e* (fig. 1, H,); in new untreated stave flumes of either redwood or fir (fig. 1, F); and in the smoothest of concrete pipes when used as flow lines, i. e., covered flumes, for reasonably straight reaches; but it should not be used as a design value to hold throughout the life of the structure.

$n=0.012$. In the old list attributed to Kutter this value is given "for unplaned timber." The data upon which this recommendation was based were obtained largely from "test channels" of unplaned lumber, presumably quite new (*10, pp. 169–172*). More recent experimental data indicate that this value is found only in new, untreated, planed-timber flumes operating under excellent conditions. No value of n less than 0.015 should be used for long, unplaned timber flumes. That the surfaces of planed and unplaned plank flumes tend to become identical with respect to frictional effects should be borne in mind. It is difficult to determine as to the initial distinction after a flume is 10 years old or more.

A value of 0.012 will be attained quite frequently in metal flumes of types *d* and *e*, especially when they are unpainted and reasonably straight, and also in stave flumes. It is only occasionally found in rectangular concrete flumes without cover. It may be used in the design of such flumes where alinement is straight and the length is less than about 500 feet. For such short flumes a slight lack in the value of n will be overcome by an additional effective slope of the energy gradient, developed by heading-up in the canal above the flume, or by raising the water in the upper end of the flume. For structures that unquestionably would be classified as "long flumes" (p. 8) 0.012 is too low for design except in especially favorable cases. For such flumes any lack in the value of n could not be compensated by any feasible changes in effective gradient. (See discussion regarding flume no. 104 on p. 78.)

This value of n may be used for short, straight flumes of unpainted metal, surfaced lumber, or smooth concrete, where algæ or insect growth will not prove troublesome. The longer the flume, the less conservative is this value of n, provided that it is not too long to be classed as a short flume.

$n=0.013$. This value of n is about the lowest that should be used for conservative design of a long flume, regardless of material. Where previous experience indicates there will be no trouble with algæ or insect growths, this value is applicable to reasonably straight flumes as follows:

(1) Metal flumes of types *d* and *e* so painted that a continuous glossy surface results.

(2) Poured-concrete flumes where oiled steel forms are used for the sides and a smoothly trowelled bottom will surely be attained. Excellent sides have sometimes been discounted by rough bottom finish. (See nos. 1–9, 16.)

(3) Treated wood-stave flumes where experience indicates that little or no algæ or insect growth will reduce the capacity or where brushing or chemical treatment is anticipated, thus insuring a reasonably clean flume.

(4) Covered, surfaced lumber-plank flumes without battens, or with tight battens which have been fully included in area and perimeter computations.

All oakum used in calking, asphaltum paints, vertical battens, metal compression rods, concrete form fins, etc., must be smoothed down so that the obstruction to flow due to their use is reduced to a minimum. Projecting roughness rapidly raises the value of n.

$n=0.014$. This is an excellent value for conservatively designed structures of painted metal, wood stave, or concrete flumes under usual conditions. It will care for reasonably sinuous alignment, slight algae growth, slight insect growth, or slight sand and gravel deposition. However, if 2 or 3 of these reducing influences are to be present a higher value of n should be used. Also, this value is applicable to uncovered plank flumes constructed of surfaced lumber with battened cracks. In computing R, the edges of the battens should be added to the perimeter of the net-water section and the area reduced by the sum of the areas of the batten cross sections. Silt and moss deposits on top edges of these battens often cause material reduction in capacity. In fact battens can cause reduction in capacity in so many ways that they should be avoided wherever it is feasible. Examination of the insides of many old battened flumes demonstrates this very forcibly.

This value is applicable to gunite flumes "shot" from the inside but surfaced with a troweled coat of cement mortar. This type of construction is adaptable to bench flumes having reasonable curvature. (See no. 15, pls. 9, C, and 12, A.)

This value can be used for flumes that would call for an n of 0.013 as regards surface and water, but with excessive alignment curvature, say 300 degrees per thousand feet of flume.

$n=0.015$. Applicable to sinuous flumes, subject to algae deposit; smoothly painted metal of types *d* and *e*; concrete with best grade of workmanship on sides but roughly troweled bottom; wood-stave flumes with well bedded algae and moss growth.

It is applicable to clean bench flumes of gunite shot from the outside against smooth wood or steel forms with the floor left as shot from the gun (pl. 1, C); plank flumes of unsurfaced lumber in typical mountain canyons with changes of direction made in short angular shifts, some gravel debris on the bottom, and maintenance repairs with scraps of old lumber, and for metal flumes of *b* type with shallow compression member projecting into the water prism, under alignment and water conditions that otherwise would give smooth metal flumes a rating of 0.013. Bottom deposits of sand and gravel become noticeable with a value of $n=0.015$ for flumes that would otherwise be in the 0.013 or 0.014 classes, and such influence should be considered if the bed load is not to be diverted by some form of sand trap.

$n=0.016$. This value is applicable to the straightest and best of metal flumes with heavy compression bars projecting into the water prism at the sheet joints (fig. 1, H, *a*). This type was extensively used up to about 15 years ago. It is not manufactured at this time but many of these flumes are still in operation. Excellent concrete and metal flumes have acquired this value after a few months' accumulation of algae and caddis-fly cases (nos. 10 and 105).

The effect of sand and gravel commences to become acute with about this value of n. Such bed load should be trapped out of a long flume. Rapid erosion, as aided by the abrasive, will roughen the bottom of a concrete flume and wear out the bottom of a metal or wooden flume.

Flumes originally rating 0.013 or more will take on this value in waters that develop lime incrustations. Heavy silt-and-moss accumulations of a leathery texture also tend to make this value applicable.

$n=0.017$ and higher. The highest values of n are applicable to metal flumes with heavy compression bars projecting into the water prism, with alignment or other conditions conducive to extra friction losses. Where higher values of n than 0.017 must be used, the flumes usually form short structures between tunnels or span gulches between sections of rock-cut or cobble-bottom canals. Ravelings from the tunnels or rock cuts or sand and boulders collect along the flume bottom to such an extent that it loses much of its identity, from a capacity standpoint, as a flume and is quite likely to take a value of n approximating that of an ordinary earth canal, perhaps from 0.023 to 0.025. The position of a short flume bottom as compared with that of the canal sections at the end of the flume has much influence on the amount and permanence of such accumulations. A short section with elevated bottom (pl. 14, C) will usually remain quite clean, while a depressed bottom is almost sure to acquire a deep bed of sand and gravel. Lime accretions, while rare, should be considered.

RELATION OF FLOW STAGE TO SOLUTION OF FORMULAS

A question that may occur to the reader is this: When a problem is solved, say by the Kutter formula, does the resulting water section have the properties of the streaming, low-velocity stage or the shooting, high-velocity stage? Solution of such problems may give results at streaming, shooting, or critical stage without recognition of the three conditions of flow, taking into account values of n, R, V, and S, only. Other tests, such as are shown in formulas 19 to 27 inclusive, must be applied to determine the stage of flow. As a rule, however, the alternate stages are so far apart that there is little chance of confusing the results. The alternate stages are merely two very dissimilar problems, as interpreted in flow formulas, one at a reasonable velocity in a good-sized water prism flowing, usually, on a slope of less than 2 feet per thousand. At the alternate stage the velocity is noticeably high, the size of the water prism, represented by R, is usually much less than ordinary examples cover, and the slope is immediately recognized as a steep incline. In general, a much steeper slope is required to maintain the same flow at shooting stage than is the case for streaming stage, since the value of R is smaller and the velocity is greater.

FREEBOARD

The effective distance between the surface of the water in a flume and the top edge of the side walls or the bottoms of cross bars is called the "freeboard." For purposes of design, it is usually a strip of constant width. As encountered in actual operation, it is generally variable over a wide range. Formulas used by different authorities give results that make material differences in the computed capacity of the structure. Obviously, a great difference in capacity, and hence in size requirement, can be obtained by a difference in the assumptions for various types of flume. In the determination of freeboard for a given flume, the following items must be considered:

(1) The velocity of the water.

(2) Alignment of the flume.

(3) Effect of wind due to exposure and water surface area of the flume, the effect being roughly proportional to this area.

(4) Amount and type of trash that winds may blow into the canal.

(5) The hazard of high winds blowing upstream in the flume; both checking the velocity—thus making the water deeper, and developing wind waves.

(6) The injury the flume may sustain in case water does splash over the sides.

(7) The protection against overloading afforded by siphon or by lateral spills.

(8) The change in stage due to "checking up" water to make a delivery to high land below the flume.

(9) The change in stage due to the change in the value of friction factors.

(10) The type of flow likely to hold at maximum capacity, mild, critical, or fast.

(11) The possibility of flow changing from one type to another.

(12) The feasibility of increasing the freeboard locally to care for bottlenecks that develop on all long, open conduit systems.

Authorities do not agree as to the items that should enter a formula for freeboard. The United States Bureau of Reclamation recognizes depth, diameter, and velocity head.

For semicircular flumes, minimum freeboard $= 0.1\ D\ (0.9 + h)$ (35)

One company with extensive experience in metal flumes suggests

for semicircular flumes, freeboard $= 0.06\ D$ (except for velocities near the critical) (36)

Etcheverry (*8, v. 3*) bases his suggestion wholly on water depth.

For flumes, freeboard, in inches $=$ depth of water in feet $+ 2$ (37)

The writer suggests the following formula as a base, the results to be adjusted by consideration of the modifying factors listed above:

Freeboard $= 1$ velocity head plus 0.25 feet $= h + 0.25$ (38)

with a maximum of 2 feet except under extraordinary conditions.

In operation this would yield a freeboard of 2.0 feet for a velocity of 10.6 feet per second without further increase; of 15 inches for $V = 8$; of 1 foot for $V = 7$; of 6 inches for $V = 4$; of 4 inches for $V = 2$.

Sharp unrounded angles, need for a definite maximum capacity, and uncertainty as to amount of algae or insect troubles to be anticipated, are most common reasons for materially increasing the designed freeboard.

In a discussion of freeboard, it is again advisable to recall that the present-day stave flume, by virtue of construction, has a definite freeboard provided above the mid-diameter and in most cases the complete lower half of the circle can be used for the water prism (*3*).

EFFECT OF SHAPE

The cross-sectional shapes of flumes are more or less dependent on the materials used in their construction. From the standpoint of material to be employed they may, therefore, be classified with the associated characteristic shapes, as follows:

The concrete flume usually with the rectangle, modified by fillets in the corners and with a bottom slightly dished; occasionally with the trapezoid having steep sides; and rarely with the semicircle or hydrostatic catenary.

The sheet-metal flume with an approach to the hydrostatic catenary; occasionally with vertical sides and dished bottom.

The plate-metal flume with the rectangle.

The wood-plank flume with the rectangle and in a few cases with the triangle. (Where the primary object is the transportation of logs and lumber, the triangle is generally used.)

The wood-stave flume with the true semicircle for the water section with freeboard extending into the upper half of the circle. Very large stave flumes of elliptical form have been considered, or with one radius for the bottom and sharper curves for the sides.

Other elements being equal, the capacity of a flume is greatest for the maximum value of R. In rectangular flumes this condition occurs where the width is twice the water depth. For curved shapes, the semicircle gives the maximum value. In wood-plank flumes of rectangular shape and of highly variable load, the flume is often made relatively wide and shallow so that the flow can vary materially without exposing an additional crack between sideboards. If these cracks can be kept wet the leakage is small, but if they dry out heavy leakage may result for a long period of time after the flow stage is raised to cover the crack. In other words, hydraulic properties are sometimes sacrificed for operation improvement.

The freeboard in a stave flume is especially effective because the inward curves above the mid-diameter tend to throw waves back into the flume rather than allow them to slop over the sides. Flumes of shapes and construction that do not require crossties are, of course, not subject to the entanglement of tumble weeds and other floating brush. Difficulties due to trash in the crossties are not frequent, but should be considered in some localities. The smaller flumes are more liable to such trouble.

Capacities of sheet-metal flumes have generally been computed as portions of a semicircle. For preliminary calculations for a metal flume, at full capacity, the diagram in figure 4 may be used. Actual areas for surface elevations giving a freeboard of 0.06 D are about 1 percent less than the areas given. This reduction is due to the catenary sag (fig. 1, and table 4). This is a maximum for depths from about one third to one half full so it does not have its greatest effect near maximum capacity. The sag should be considered in all field tests; the surface chord and the depth not being used to compute a segment of a circle whose diameter is the nominal diameter of the flume. Actual areas, perimeters, and resulting values of r are slightly different.

TABLE 4.—*Typicals sags, indicating distortion from the circle, as measured in metal flumes of various sizes for various depths*

Reference no.[1]	Flume number equals length of sheet	Diameter	Water depth		Sag below circle		
			Measured	Percentage of diameter	Distance	Sag as percentage of depth	Sag as percentage of diameter
	Inches	*Feet*	*Feet*	*Percent*	*Feet*	*Percent*	*Percent*
---------	108	5.73	1.11	19.37	0.10	9.01	1.75
---------	108	5.73	1.87	32.63	.12	6.42	2.10
---------	120	6.366	1.45	22.78	.15	10.34	2.36
---------	120	6.366	2.46	38.64	.18	7.32	2.83
101	132	7.003	3.16	45.12	.28	8.86	4.00
---------	156	8.276	2.00	24.17	.20	10.00	2.42
---------	156	8.276	2.35	28.40	.20	8.51	2.42
---------	156	8.276	2.50	30.21	.20	8.00	2.42
---------	156	8.276	2.72	32.87	.17	6.25	2.05
103	180	9.55	4.06	42.51	.40	9.85	4.19
103	180	9.55	4.23	44.30	.41	9.69	4.29
176	180	9.55	5.03	52.67	.41	8.18	4.30
98	204	10.82	2.24	20.70	.48	21.43	4.44
98	204	10.82	2.59	23.94	.46	17.76	4.25
98	204	10.82	3.45	31.86	.59	17.10	5.45
166	348	18.46	8.88	48.10	.68	7.66	3.68

[1] See table 2

EFFECT OF CURVATURE

In the opinion of the writer present ideas regarding friction losses incident to curvature, over and above those occurring in a straight channel, and likewise the encroachment on freeboard due to superelevation of the water surface on the outside of a curve, are subject to modification.

ADDITIONAL FRICTION LOSS

It has long been realized that little additional information regarding losses was to be gained from a few scattered curves considered in connection with short tangents included in the same reach of flume. Prior to the construction of Tiger Creek conduit (nos. 39 to 54) experiments on flumes of similar construction, but on reaches less than one half mile in length, brought out the fact that the effects of each curve were extended both upstream and downstream into the adjacent tangents so that no excess local loss around the curve was indicated by the energy gradient. The water surface showed superelevation on the outside and corresponding depression on the inside, but the energy gradient throughout the reaches tested was a remarkably straight line. This gradient was developed by adding the velocity head, h, for the mean velocity, v, at each section, to the mean elevation of the water surface, Z, on the two sides of the concrete channel. The water surface was taken at intervals of 50 feet on tangents and 25 feet or oftener on sharp curves.

The original device based on the principle of the surface gage shown in plate 10, A, was developed by A. W. Kidder, engineer for the company owning the canal, and the writer, for the purpose of making the measurements taken. From these tests on curves and bends and all others that could be collated, it was decided that a minimum radius of 75 feet could be used on Tiger Creek flume. Results have justified this decision. The conduit was placed in commission in June 1931. Just before running the first water, five reaches, each about 4,000 feet long, scattered along the 20-mile conduit, were selected for making tests. A level party painted black spots on the top inside edges of both concrete walls every 50 feet on tangents and 25 feet on curves and tied the elevations of the spots and of the flume bottom to the bench marks in the scheme of levels finally adopted for construction of the conduit.

In August 1931 tests were made to determine: (1) The friction factors near the feeding reservoir after 2 months' accumulation of algae growth; (2) the change with distance from reservoir in these factors; (3) the effect of curvature on the friction factors throughout lengths having excessive curvature, in comparison with those having minimum curvature within any one of the reaches; and (4) the effect of curvature on the water surface and character of flow. In November 1931 tests 50, 52, and 54 were conducted after the algae had disappeared from the flume. In the spring of 1932 tests 46 to 48, inclusive, were made before the algae started growing. To accomplish a complete test for any one reach while the flow was held constant, it was necessary to develop equipment and methods so that a large number of precision measurements could be made from the points spotted, down to the average water surface, within the period of time necessary to make a measurement of the flow, Q, by the multiple-point method, i.e., in a large number of verticals, with points in each vertical at the surface, 0.2, 0.4, 0.6, 0.8 depth, and bottom. With two hydrographers about 2½ hours were required for such a measurement.[13]

[13] The writer believes the equipment, field conditions, and methods adopted for the measurements on Tiger Creek conduit were such as to yield data among the best available resulting from tests on values of n, flow around curves in large channels, extent of nonuniform flow, and relationship between curve losses and extent of curvature for a particular channel. In these measurements the Pacific Gas & Electric Co., through its engineers and load dispatchers, cooperated to the fullest extent.

On reach 39, a number of observations of water surface were made by two gage readers, one on each side of the flume, traveling down the structure on the carriages shown in the inset of plate 13, B. These carriages could be operated by boards or paddles held in the flowing stream, or they could be pushed by an assistant walking along the ground at the flume side. The latter method proved fast and also sure in stopping, so each spotted point was within easy reach of the gage reader. About 30 seconds were allowed for the water in the stilling well of each gage to become settled before taking the reading. One engineer on the ground kept notes for both gage readers. The elevations of the points spotted were already available and listed on the field sheet. The gage reader announced the "constant" distance, measured to tenths of a foot on a rod so graduated, from the spot, (*P*) (pl. 10, A) down to the zero point of the gage in the stilling well. The hook-gage (*a*) reading, to the nearest hundredth foot, was next given to the note keeper. The final elevation of the damped-down average elevation of the water surface at the side of the flume at the given station equaled: elevation of spot − constant + hook-gage reading.

In further computations the mean of the surface elevations at the two sides of the flume was taken as the water-surface height. To this mean elevation was added the velocity head for the mean velocity of the water at that section, the sum being the elevation of the energy line at that station. The rate of fall in the energy line was used to compute the friction loss. Previous measurements had determined the fact that there were no appreciable differences in the standard flume section from station to station.

For purposes of study, profiles of the bottom, water surface at right and left sides, and the energy line as determined above were plotted on cross-section paper. There was no tangent on the flume of sufficient length to be taken as the straight-line criterion to serve as a base in studying the curvature. However, it was observed that there were definite but unequal inclinations of slope in the energy line when long reaches were plotted. Uniform flow did not exist anywhere along the 20 miles of flume, so far as close measurements showed. A mass diagram of the curvature was plotted under the profiles mentioned above, using the same horizontal scale as for the profiles but using total degrees of curvature (regardless of radius) for the vertical scale. Thus sharpness of curvature was shown as a steep line with length of curvature as the length of this inclined line, tangents showing as horizontal reaches on the mass diagram. By this method, for the first time known to the writer, the steepness of the energy gradient appeared as conforming to the steepness of the mass diagram of curvature. For example, nos. 41 and 42 of table 2 refer to adjoining portions of conduit with the same conditions holding except the total amount of curvature per 100 feet of flume. Note the difference in the friction factors. Other reaches in Tiger Creek flume may be compared in the same way.

The curves in concrete flumes, so far as the writer is informed, have always been made with straight forms. This method gives a curve composed of short, straight sections. On Tiger Creek flume the form units were 6 feet long (measured lengthwise of the flume). On the Tieton flume (no. 10) they were but 2 feet long, and on the Umatilla conduit (no. 76–79, inclusive) they were 10 feet long. Hopson (*15,*

p. 182) ascribes much of the difference in the roughness of the water in these two flumes to the difference in the lengths of the form units.

Wood-stave flumes can be built on true curves, but the minimum radius is restricted. For metal flumes curves are made with special gore sheets, the result, as in concrete flumes, being a series of short tangents rather than of true curves.

To suggest the friction factor for specific rates of curvature—that is, sharpness of curves—is not yet feasible; but the following tentative suggestion as to retardation due to curvature in a rectangular flume appears to be warranted from a comparison of the differences in retardation with the differences in total angular amounts of curvature. Using the slope of the mass diagram of curvature—that is, the number of degrees of curve per 100 feet of flume—an average increase of about 0.001 in the value of Kutter's n for each 20° of curvature in 100 feet of flume is indicated. In several parts of Tiger Creek flume the curvature approached 500° in 1,000 feet of flume. The suggestion made would call for an increase in such reaches of 0.0025 in the value of n.

Plate 10, B shows about 430° of curvature in a length of 780 feet. All of the curves have radii of 75 feet except the first one, which turns to the left with a radius of 140 feet, followed by one turning to the right with a radius of 90 feet. A large amount of curvature was necessary for any feasible flume down the canyon. It was finally decided that it was more important to keep the flume on the excavated portion of a bench, following the contours rather closely, than to reduce the curvature at the expense of foundation stability.

SUPERELEVATION OF WATER SURFACE ON OUTSIDE OF CURVES

The Tiger Creek flume [14] tests clearly indicate the following tendencies:

(1) The average water lines were elevated on the outside and depressed on the inside in about equal amounts, the maximum difference shown by the averaging gages being about 0.4 foot (see item 10 below) for a mean velocity of about 6.7 feet per second.

(2) The difference appeared to depend upon the length more than upon the sharpness (represented by the shortness of radius) of the curve, gradually increasing to about the middle of the curve.

(3) Beyond the middle of the curve, the amount (see (1)) depended upon the direction of the next curve; i.e., was subject to backwater conditions.

(4) If the next curve was a reverse, starting at the end of the curve under consideration, the amount decreased so that the surface was without elevation or depression at about the point of reverse curve. The surface warp reversed at the same point as the curvature.

[14] The deductions are made from exhaustive tests on a rectangular flume 14 feet wide, with a water depth of nearly 6 feet; Q ranging from 516 to 540 second-feet, and mean velocities, less than the critical of from 6.4 to 7.4 feet per second. Most of the curves have radii of from 90 to 140 feet, with occasional ones of 75 feet, and a few flatter ones with radii of several hundred feet. As shown in the views of this flume (pls. 13, B and 15, A) water took the sharp curvature without undue disturbance; the energy gradient showed no local sharpness of incline around reverse curves aggregating nearly 200° of curvature with many individual curves of more than 90°. The effect of excessive curvature was manifest in slightly greater slope of the energy line for long reaches having a steep incline for the mass diagram of curvature. (Note values of energy slope on table 2, items nos. 39 to 54, inclusive, as compared with each other and with the bed slope.) A general deduction to be carried over to the discussion of freeboard (p. 56) is that slightly additional freeboard is needed on the outside of long curves; but it is also possible that the standard freeboard may not be sufficient on the inside, owing to the choppiness of the waves. It has been stated that overtopping occurred on the inside of Tieton-flume curves, rather than on the outside, until reconstruction gave additional freeboard all along the flume (*15, p. 179*) (pl. 9, B).

(5) If a tangent intervened between curves, the amount of superelevation decreased toward the end of the curve, but as a rule gradually tapered out in the tangent, the increase in elevation remaining on the side following the outside of the curve.

(6) If the intervening tangent was very short and the following curve had the same direction as the preceding one the decrease was noticeable but not complete, the superelevation increasing again as the second curve was entered.

(7) If the intervening tangent was relatively long (200 feet was a long tangent on Tiger Creek flume) and the following curve trended in the direction opposite to that of the preceding curve, the surface became quite flat before the following curve was entered. This confirms the conclusion that the best current-meter station on a sinuous canal is near the lower end of a relatively long tangent between curves having opposite directions.

(8) In (7), above, if the following curve is in the same direction, the amount decreases slowly, but not entirely, through the tangent and again increases as the following curve is entered.

(9) On the outside of a curve, the average water surface was not more than about 0.1 foot at variance with the amplitude of the waves. This is shown in the flatness of the fountain head rising upward on the outside of the curves. This deduction also applies to sharp curvature in a semicircular section as exemplified by Tieton flume (pl. 9, B). An example in a rectangular flume is plate 9, C.

(10) On the inside of the curves, the surface is more choppy than on the outside. This roughness prevents taking advantage of the lowering of freeboard indicated by the average depression of surface. The variance between crests and valleys of waves was about 0.6 foot (apparently amounting to about one velocity head, h). On very sharp curves, the maximum forward velocities were on the insides of the curves.

(11) The plotted energy line seldom varies locally more than 0.05 foot from a straight incline. On these long reaches, it was slightly steeper over reaches of 1,000 feet or more where the corresponding mass of curvature was relatively excessive. The water line was exceedingly irregular; therefore the deduction is obvious that tests with a few readings of the elevation of water surface, perhaps on one side of a flume only, with but moderate curvature, are of little value in determining the effect of curvature on the retardation factor.

EFFECTS OF ALGAE AND INSECT LIFE

The capacity of a smooth flume is greatly reduced by either accretions of algae or various types of insect life. This reduction cannot be considered as an effect of age since flumes but a few months old sometimes show reductions in capacity of from 10 to 20 percent during midsummer and a re-gain with the approach of autumn.

Several varieties of algal growth become manifest in spring, reach the peak stage in late July or August, and gradually decrease in September. The green algae mentioned by Taylor (*27*) and Tiffany (*29*) occupied a band on each side of the flume, extending but a few inches below the water surface, but the growth appeared to influence the value of n about 0.002. In 1931 the writer observed a similar light-green growth on a stave flume, no. 305, but it appeared to cover the periphery as deeply as could be observed. The brown algae in Tiger Creek flume covered the whole interior of the flume except where it was scoured off by traveling detritus. This appeared to be the case also near the intake end of Bowman-Spaulding flume (no. 105).

Copper sulphate treatments, in heavy but well-separated doses, appear effective in the removal of both green and brown algae. Sacks suspended in slow-flowing water, with about 70 pounds of copper sulphate per 100 second-feet of water, have given dosage that was effective for several weeks. Different forms of algae or moss require different amounts of copper sulphate and different treatments. The best way to determine cost and effect still appears to be by experiment.

There is some division of opinion as to the prevention of algæ by roofing the flume. In southern California, moss and algal growths can be fully controlled in that way. On the other hand, Hopson (*15, p. 187*) cites many localities where the flow through covered channels was affected by such growths. Their influence, in the observation of the writer, extends even to very cold waters having temperatures ranging from 35° to 45° F., and is particularly effective at the upper ends of open flumes receiving water from deep reservoirs. Many Western types of algæ must have sunlight, and a roof over a flume appears to prevent the growth. High velocities do not prevent it. Abrasive material—granite gravel, basalt ravelings, etc.—will maintain a clean streak, free from algæ along the bottom of a flume and this streak will move to the inside of curves, reaching to the top edges of semicircular flumes and well up on the sides of rectangular flumes. The growth is most troublesome in the clearest of waters and is least so in muddy waters, owning possibly to abrasion as well as the partial exclusion of sunlight in the latter. Heavy coats of algæ and moss are found in chute flumes that have velocities as high as 40 feet per second. It is therefore evident that no scouring effect whatever can be relied upon from rapidly flowing, clear water. The material of the structure itself may show scour when the growth does not.

A bed load of sand and gravel traveling with moderately high velocities may scour algæ from streaks along the bottom where still greater velocities simply lift this load and scatter it throughout the water prism, without causing appreciable scour. This possibility is often overlooked in discussions of scouring velocities.

Algæ are combated in various ways. If the coating extends but a short distance below the water line, the water surface may be lowered for a few clear days and hot sunshine will dry up the growth so it will drop or be flushed off when the flow is increased, but it soon grows again. If the whole perimeter of the flume is affected, water may be turned out for 3 or 4 days and the coat will dry up and scour out on the return of the water.

A long flume near Yakima, Wash., was scoured every few days by using a weighted V structure, with a bicycle wheel at the apex, behind which, at the open end of the V, was a set of rough brushes fitting the flume section (*5*). Tests nos. 102 and 308 were made immediately after a trip of this device down the flume. The earlier tests in 1921 (nos. 101 and 307) were made when the flume was in need of cleaning. The retardation of the water is shown in the values of n, and likewise in the total flows (Q) that held for the two tests. In both cases, the capacity of the flume was being crowded to its utmost. It is said that the passage of water-logged willow or other brush is somewhat effective in removing algal and insect growths from flumes. Patrolmen should accompany the scrubber or the brush to see that a downstream movement is continuous.

The tests on Tiger Creek conduit (nos. 39–54) show that the influence of algæ was greatest near the feeding reservoir, and that but little effect was in evidence some 15 miles away. Likewise, the tests on reach 48, made in April 1932, in a flume entirely free from algæ, may be compared with tests made on the same reach in August 1931, at a time when the algal growth was present, although the flume had then been in operation for only about 3 months. It is suggested that decrease in flow due to algal growth always be provided for, unless experience in the given locality with the available water indicates that little effect is to be expected. More often than not the growth will appear.

Since the effects of algal growth and insect life are seasonal, flume capacity may be considered in connection with the demand or supply of water. The water supply of a project without storage may be obtained by direct diversion and reach its peak before the greatest effect of algæ and insects is felt so that the flume has capacity for the reduced amount of available water by the time the growth has its greatest influence. Likewise, flumes on feeder canals serving reservoirs with storage rights, as is frequently the case in Colorado, will be operated during the nonirrigation season, in months free from stages of algæ or insect life capable of causing operation troubles.

Among the various insects that affect the capacities of flumes are the caddis fly, common in the Pacific Northwest, and the cocoons and pupa shells of the black fly of the genus *Simulium* (*30*).

The caddis fly lays its eggs in or above clear, rushing water of mountain streams. The larvæ ingeniously make housing cases of sand grains, sticks, or pine needles. As encountered in flumes of the Northwest, these cases are attached to the flume surface and are so excessively rough and so numerous that the value of n is raised from about 0.013 to 0.015 or more, a reduction in capacity of some 15 percent. One species that is much in evidence makes cornucopia-shaped cases, about one-fourth inch in diameter at the larger end. The writer has found these so thick on a flume wall that the hand would cover 20 or more shells at one time. The pupal or inactive stage of the life cycle of the insect is begun by the mere forming of a covering over the opening of the case. Here the pupa remains until time to emerge as the imago, or perfect insect. The caddis fly cases may contain the pupæ all winter; thus making the effect on flume capacity extend over a long period.

Several species of the 2-winged flies (order Diptera) spend their larval and pupal stages in fast-flowing water and during these stages are commonly found in flumes. The larvæ (soft worms without shell covering) are found in great numbers, the hand possibly covering 50 to 100 individuals, and affect the flow near the water surface. The pupal stage is spent in large clusters of jellylike, turtle-shaped domes, firmly attached to the sides of flumes, in the most turbulent flows. These insects appear to require both the moisture and also the aeration of rough water for survival.

In the larval stage, these insects may be reduced by brush treatments. Withdrawing the water for a few days is but partially effective. Entomologists suggest that the insects may be combatted at the egg-laying stage by suspending an oil-soaked rope in the water, or by painting a strip along the range covered by the water-surface line with some asphaltum mixture that would be a repellent to the adult female when she attempts to lay her eggs at the water edge of a flume. If effective, the latter method appears the more practical of the two.

PROBLEMS IN CAPACITY DESIGN

Flume problems come to the practicing engineer in two rather distinct phases: (1) With reference to a major or minor conduit in a system being projected; and (2) as a renewal or an emergency repair item in a system otherwise fully established.

Nearly all capacity problems for conduits of any type have a desired maximum quantity Q as a fixed item. If the channel slope is also fixed a relatively few trial dimensions based on a chosen value of n for a given type of flume will yield one answer, while another shape characteristic of another type of flume will yield a different answer.

Usually, in a project under office study, after preliminary field work, there is a reasonable amount of elasticity in the adjustment of local grades and dimensions for any flume under consideration. Where a new flume, usually of a different type and material, replaces one that is worn out or too small, the total fall available to the new structure is fixed within a narrow range. It then remains to divide this total fall into its component elements of entry loss, friction loss, and outlet loss. When the flume is to replace a washed-out reach of excavated canal, usually along a steep hillside there appears available for total fall only the friction loss that was necessary in the original canal. Thus, in the same distance, the loss incident to a value, let us say, of $n=0.023$, for the excavated canal, is available in a flume structure for which it is decided that a working value of $n=0.014$ will be sufficient.

Usually this substitution of a flume for an earth or rock-cut canal is so short that the head salvaged by improving the surface is insufficient to care for inlet and outlet losses plus any material friction loss incident to higher velocities in the flume. The final results of study usually determine a flume with velocities increased but a minimum, sectional shape changed as little as feasible, and the canal banks above the flume structure raised to allow required heading-up to overcome the extra losses in the flume. Another way to develop the additional fall required is to line the canal above and below the flume structure for sufficient distance to salvage head by the improvement of canal surface. As a rule the lining is also required to stop the seepage loss that caused, in part at least, the canal slip which necessitated the flume in the first place.

Replacements of worn-out structures, especially of old plank flumes, usually are based on a larger capacity. Many plank flumes were constructed on irrigation systems on the assumption that some years must elapse before peak requirements in capacity would be necessary and by that time the original flume would be worn out and a larger capacity flume built. Thus, the flume that has been taken for an example (p. 67) is a third-generation flume, the sequence being, (1) a small wooden box flume, (2) a larger one (reported as no. 64 in (*24*)) and (3) a wood-stave flume of large size. A definite heading at the river and a definite position of the excavated canal below necessitated the use of the original slope of 0.002 for the final flume. Thus, the normal velocity in each succeeding flume was higher than that in the one preceding, until a normal velocity approximating the critical was finally obtained (p. 67).

To facilitate solution of flow problems by the Kutter or Manning formulas the diagrams in figures 3 and 4 are offered.

ESTIMATE DIAGRAMS

Figure 3 gives a solution for general problems involving the Kutter formula. The use is best explained by an example. The dashed lines show that in a channel with hydraulic radius, $R=2.6$ and with an assumed value of $n=0.015$ and a slope of 0.00125 the velocity will be about 6.7 feet per second. The quantity of flow, Q, is then equal to AV. This diagram can be used for design of flumes of any shape and also for canal sections at the ends of the flumes. For very flat slopes interpolate between guide lines, which are split for divergent values of n.

Figure 4 is offered for solution of problems relating to semicircular flumes with radii from 1 to 10 feet, for any depth from 1 foot to full depth (equal to radius) at mid-diameter. Depth is given both in feet and in percentage of radius. Slopes range from 0.0001 to 0.100, velocities from 1.5 to 40 or more feet per second, and values of Manning's n' from 0.008 (for the trajectory in chute flumes) to 0.020. The Manning formula allowed straight-line values of n' whereas the Kutter formula could not be drafted in this form to give graphical answers. For flumes the two formulas agree closely enough for graphic design purposes. Explanation of use is dashed in two examples:

(1) Using the elements for the standard example (p. 67) as shown in heavy dashes:

Enter diagram at flume-radius 6.0; thence move vertically downward to full depth line; thence horizontally to left to intersection R for full depth $=3.00$ feet; thence vertically upward to slope-line $S=0.002$; thence follow guide lines to intersection with $n'=0.012$, which occurs at $V=11.6$.

Return to intersection for R at full depth; thence to the left to area, A at full depth; thence upward to base line; thence indefinitely at 45° (to multiply).

Return to intersection of n' and $V=11.6$ feet per second. By the scale at the left convert V to 11.6 directly above, by the arrows; thence to the right, intersecting the indefinite 45° line at quantity $Q=656$ second-feet.

Example (2) (lightly dashed lines) shows elements for a no. 192 metal flume with projecting compression bands for which a value of $n=0.017$ is chosen. What are the approximate elements for this flume with a slope of 0.00065 when 70 percent (in depth) full? In the lower right-hand corner enter diagram at full depth line for no. 192 flume; thence downward to 70-percent full line. (This could also have been found for an assumed equivalent depth of 3.6 feet, shown by the scale to the left.) Thence to the left intersecting R for 70-percent depth and on further to intersection of area, A for 70-percent depth: thence upward to base line: thence draw line of

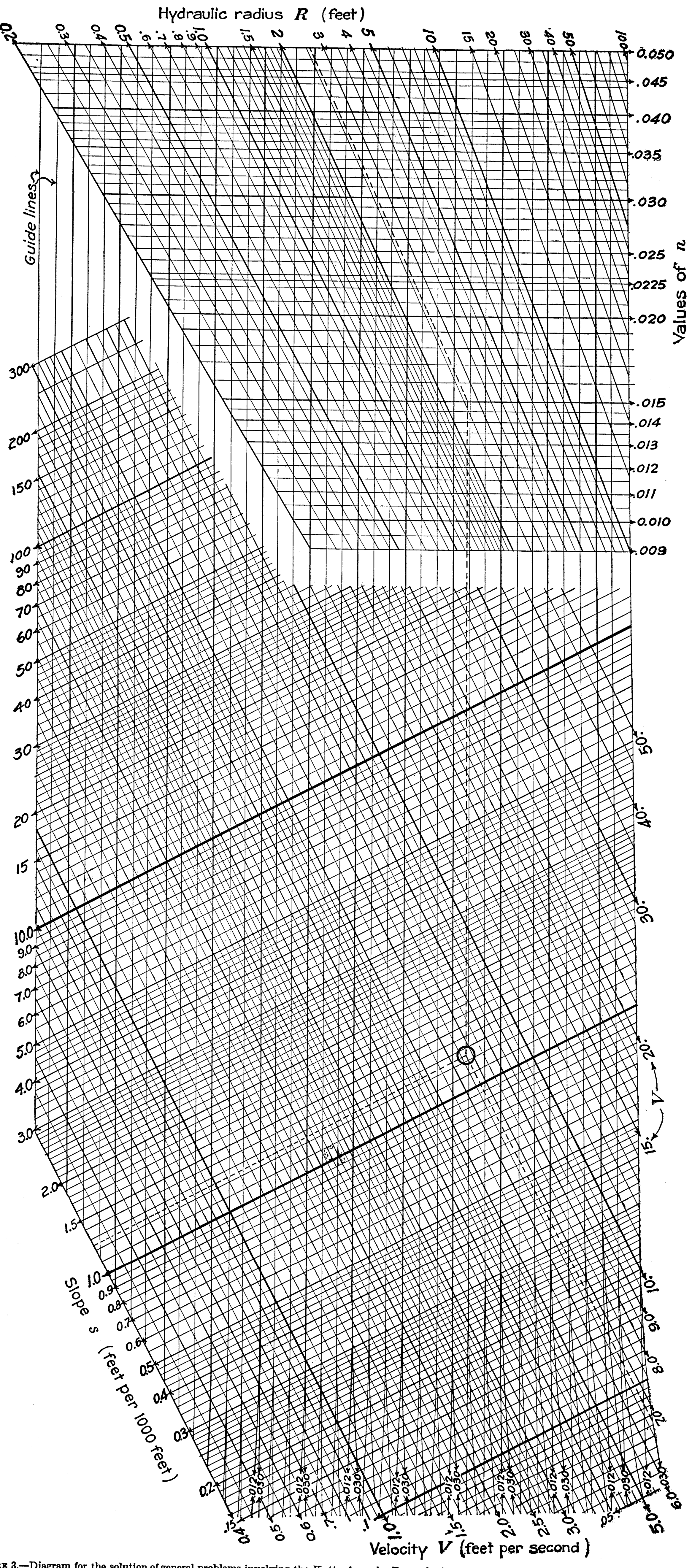

FIGURE 3.—Diagram for the solution of general problems involving the Kutter formula. From the intersection of R and n follow the guide lines to the intersection of S and V, or from the intersection of S and V follow the guide lines to the intersection of R and n. For example shown in dashed lines, see page 66.

4540°—33. (Face p. 66.) No. 1.

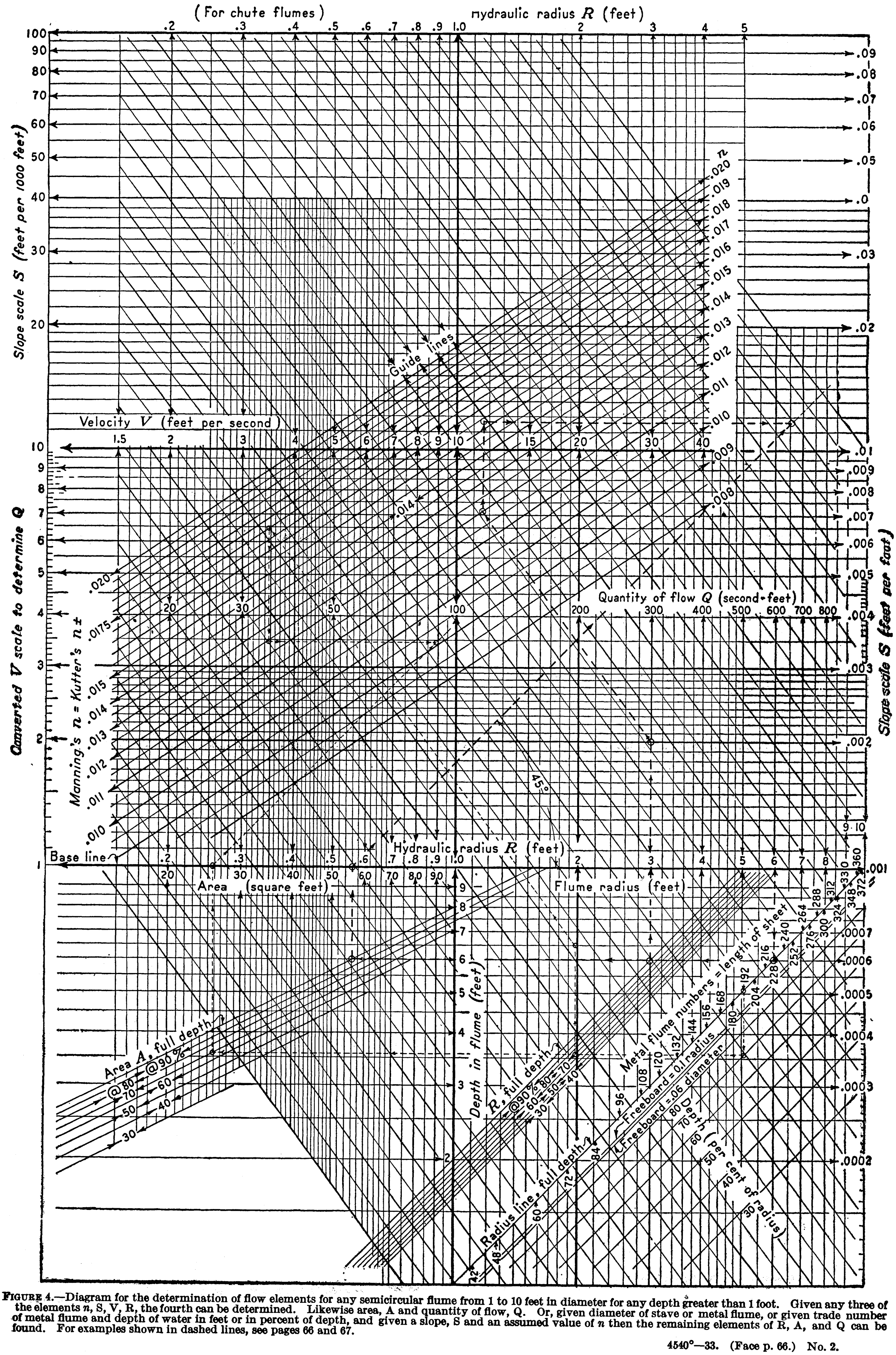

Figure 4.—Diagram for the determination of flow elements for any semicircular flume from 1 to 10 feet in diameter for any depth greater than 1 foot. Given any three of the elements n, S, V, R, the fourth can be determined. Likewise area, A and quantity of flow, Q. Or, given diameter of stave or metal flume, or given trade number of metal flume and depth of water in feet or in percent of depth, and given a slope, S and an assumed value of n then the remaining elements of R, A, and Q can be found. For examples shown in dashed lines, see pages 66 and 67.

4540°—33. (Face p. 66.) No. 2.

indefinite length, as dashed, at 45° to the right. Return to R at 70-percent depth; thence upward to slope line 0.00065, thence follow guide lines to $n' = 0.017$ indicating a velocity of about 3.48 feet per second; thence downward to convert V by scale at the left to converted $V = 3.48$; thence to the right to intersection with the line of indefinite length. This intersection shows the flow Q would be about 88 second-feet.

Problem 1. To determine the diameter of a semicircular flume to replace an old plank flume along Deschutes River, near Bend, Oreg. The given elements were as follows: Length, L, 5,820 feet; slope, S, 0.002; quantity, Q, 656 second-feet. The resulting flume design, as determined by the consulting engineers, called for a diameter of 12 feet with water occupying the complete semicircle, a value of hydraulic radius, R, equal to $D/4 = 3.00$, and a mean velocity, V, of 11.6 feet per second based on a selected coefficient of hydraulic friction (Kutter's n) of 0.012. This particular problem, taken from actual practice and with the flume finally constructed and operated, has been selected by the writer for many reasons. The length is sufficient to class the structure as a long flume. The design and construction are quite fully described in current literature, available to the reader for greater detail (*3*). The section being a true semicircle (templet determined), segmental areas for various depths are definitely determined. The value of n as selected was slightly more than the actual value when the flume was new, as determined by test (no. 301) in 1923. Subsequent tests (303 to 305, inclusive) in 1926, 1928, and 1931 determined progressive values of n with the growth of mossy algae. The value of V as computed for normal flow was quite close to the critical. By formula (20)

$$V_c = \sqrt{\frac{g\ A}{T}} = \sqrt{\frac{32.2 \times 56.6}{12}} = 12.3 \text{ feet per second}$$

against a design-velocity of 11.6 feet per second. Any short, reasonably straight reach having an actual value of n less than about 0.0113 would develop a velocity faster than the critical (fig. 7).

DESIGN ARGUMENT

As a long flume, the sectional dimensions can be developed for a normal flow without regard to details of water levels above and below the flume. Likewise, as a long flume the energy slope has the same rate as the bottom slope. After computing the dimensions, the water-surface line of the flume at maximum capacity can be placed vertically in the scheme of levels to allow for proper entry loss, for the increase in velocity from earth canal to circular flume, and for recovery of velocity head at the outlet of the flume. Only in recent years has it been customary to assume recovery of any considerable portion of the velocity-head differential.

In the design of this structure, the consultants used $n = 0.012$ for both metal flume and for creosoted-stave flume. As shown in plate 13,A, the alignment follows the usual sinuosity of a large mountain stream. In 1923 the writer would probably have used the same value of n. With the present understanding of these structures, a value of $n = 0.013$ as recommended on page 54 would be selected provided periodic cleaning of algæ growth was anticipated. Without such maintenance a value of 0.014 (p. 54) would be conservative for both treated fir and painted metal. Even this design might require occasional restoring of the original flume surface.

DESIGN PROCEDURE

Open-channel sizes are finally determined after many trials and adjustments. Using formula 2 or tables (*31*) with the value of $n = 0.012$ as chosen by the consultants, table 5 is now set up showing the possibilities.

TABLE 5.—*Trial tabulation, by solution of Kutter's formula, for conveyance of water by large flumes, problem 1*

WOOD-STAVE FLUMES

Number = length of sheet	Diameter D	Area A	Hydr. Rad. R	Velocity V	Quantity $Q=AV$	Velocity head h	Critical head h_c	Critical velocity V_c
Inches	*Feet*	*Sq. ft.*	*Feet*	*Feet per second*	*Sec.-ft.*	*Feet*	*Feet*	*Feet per second*
---------	10.0	39.27	2.5	10.4	409	1.68	1.96	11.23
---------	10.5	43.30	2.62	10.7	463	1.78	2.06	11.51
---------	11.0	47.52	2.75	11.0	523	1.88	2.16	11.79
---------	11.5	52.00	2.88	11.3	588	1.98	2.26	12.06
---------	12.0	56.60	3.0	11.6	657	2.09	2.36	12.32
---------	12.5	61.35	3.12	11.9	730	2.20	2.45	12.55
---------	13.0	66.35	3.25	12.2	810	2.31	2.55	12.81
METAL FLUMES								
240	12.73	53.96	3.07	11.46	618	2.04	2.14	11.72
252	13.37	59.49	3.22	11.75	699	2.15	2.24	12.01

The elements for the stave flumes are based on the use of the full half circle for the water section, with freeboard added in the form of additional arcs of the circle above the middiameter. For the metal-flume design, the elements must be revised as the construction is based on the full length of the metal sheet being equal to the semicircumference with the edges set at middiameter points. This requires the freeboard to be deducted from the middiameter depth. Likewise, the water section takes a form approaching that of the hydrostatic catenary, which is somewhat less than that of the true-circular section, for full water depth. The trial figures in table 5 are based on a freeboard of 0.06 D. For the metal flumes, if a freeboard of 0.06 D be used, the figures can be taken directly from the catalog of the manufacturers.

The foregoing discussion, together with a glance at figure 1, brings out the point that stave flumes and metal flumes must be computed for slightly different conditions. The stave flume uses the full half circle for the water prism, with additional arcs above the middiameter for the freeboard. The metal flume includes the freeboard under the middiameter line. Hence, the metal flume takes a nominally larger diameter than the stave flume, although the lengths of the perimeters, including freeboard, are about the same.

Reference to table 5 for the metal flume shows that a no. 240 flume is too small, and the next larger size will carry more than the design Q' of 656 second-feet. For stave flumes, the 12-foot flume (as finally constructed) will carry 657 second-feet at a mean velocity of 11.6 feet per second, while the critical velocity for the same water prism is 12.3 feet per second. These are quite close together so a very small difference between the computed and the actual values of n might cause the water to fluctuate between the streaming stage and the shooting stage. This has actually occurred with flows much less than the design Q. (See p. 85.)

Instead of resorting to tables and computations for the solution of general problems in uniform flow or of short reaches assumed as in uniform flow, it is feasible to make rapid approximate solutions by means of the diagram for the solution of Kutter's formula on figure 3 or the diagram showing solutions by the Manning formula for all necessary depths of water in a wide range of sizes of semicircular flumes, shown on figure 4. (See p. 66.)

For figure 3, given any three of the usual elements, S, V, R, and n the value of the fourth element is determined. On figure 4 the best procedure is to select a trial size of flume with an assumed free-board and trace out the velocity and quantity of flow for assumed values of S and n. In the descriptive notes under figure 4 the example given in the text above is traced, following lines dashed in the diagram. These illustrations can also be used in tracing back-water and drop-down curves as tabulated on pages 77 and 78, and used to show the application of certain characteristic curves, applying to the lower ends of long flumes and perhaps throughout the lengths of short ones.

With the hydraulic elements of the flume determined, it remains to set the flume in vertical position in regard to the canal sections at the two ends. This will be illustrated by the elements at the inlet transition.

ENTRY TRANSITION

The reader is referred to comments on entry transitions on page 12 before proceeding as follows: For flume velocities of about 8 feet per second or less, the cylinder quadrant inlet shown on plates 5 and 8, A, has been found to work very well. However, for the flume velocity of 11.6 feet per second with a velocity-head drop of more than 2 feet, it would be advisable to develop a smooth transition along the lines laid down by Hinds (*14 p. 1437*).

The flume used for the example thus far did not have an inlet transition. For any simple inlet, such as the cylinder quadrant, the position of the flume bottom can be determined as outlined below:

Regardless of the type of inlet or method of developing the transition section, the following steps outline the procedure for determining the vertical position of the inlet elements: In the following tabulation note that the actual inlet loss is relatively small and the flume velocity-head is carried in the energy content, $d+h$, throughout the structure.

In terms of Bernoulli's theorem, including loss, as given in formula 15, $k_0+d_0+h_0=k_1+d_1+h_1+h_e$, (i.e., entry loss including friction). As handled in the following computations, in terms of elevations and elements: $k_0+d_0+h_0-h_e-h_1-d_1=k_1$.

		Feet
Assume elevation of bottom of canal at transition inlet, k_0		95. 000
Assume depth of water in canal at transition inlet, d_0		5. 000
W.S. elevation in canal at transition inlet, for design Q′		100. 000
Velocity head (for mean velocity in canal of 4.00 feet per second) $=h_0$		0. 249
Elevation of energy line at transition inlet, E_0		100. 249
Velocity head for $V=11.6$ in flume	2. 092 $=h_1$	
Velocity head for $V=4.0$ in canal	. 249 $=h_0$	
Increment of velocity head	1. 843 $=\Delta h$	
Loss of head in inlet, assumed as $0.1\Delta h=h_e$		0. 184
Elevation of energy line at inlet, flume proper, E_1		100. 065
Velocity head for $V=11.6$ in flume, h_1		2. 092
Elevation of water surface, W.S. at inlet, flume proper		97. 973
Maximum depth in center of flume at inlet, flume proper, $d_1=$		6. 000
Elevation, invert or inside bottom of flume, upper end, $=k_1=$		91. 973

THE CYLINDER-QUADRANT INLET

As a substitute for the square inlet, the angular-winged inlet and, in most cases, for the expensive warped inlet, the cylinder-quadrant inlet is offered. Developed in laboratory tests, this type has been placed in field service in enough flume installations to prove its efficiency. As shown in plate 5 and plate 8, A, this transition structure is essentially a pair of circular wings, tangent to the flume sides and, for earth canals, extending into the banks as cut-off walls after curving through a quarter turn to meet the sides of the canal. The wings are simple vertical walls, easily formed with sheet metal or even narrow boards. If many flumes on one project are to be served, metal forms can be used over and over, even with different curvatures. The vertical elements of the walls are allowed to intersect the floor and the side slopes of the canal unmodified with the exception given below. For flume velocities up to about 5 feet per second, a wing-curve tangent to the flume proper with a radius equal to the diameter of the flume, is satisfactory. The point of tangency at the high-water line of the flume rather than the top edge of the flume gives the better surface. For velocities above 5 feet per second, a short straight reach between the wing tangents and the flume proper is suggested. This reach should be filled in, or "padded" with concrete so as to form fillets in the corners and change shape of prism from a rectangle to that of the flume.

For the higher velocities, the writer believes the warped transition described by Hinds (*14*) is more certain to give satisfaction. However, the hydraulics of the quadrant inlet can be studied, using various radii and various assumptions for bottom transition.

In table 6 are given the essential hydraulic data for an inlet of this type without straight section for the example flume (p. 67), in order to show that the usual conditions presented do not greatly conflict with accepted hydraulic requirements. In this example it is to be noted that the velocity is increased from 4.00 in the canal to 11.6 in the flume. This involves a higher flume velocity than is recommended as a tentative limit for this type of inlet. The drop in Z is too rapid near the flume proper, showing the necessity for a straight reach beyond the canal wings.

TABLE 6.—*Hydraulics of cylinder quadrant inlet for semicircular flume for a flow of 656 second-feet with C_e assumed as 0.10; diameter as 12 feet*

[Canal elements assumed: $b=28$ feet; $d=5$ feet; $a=164$ square feet; side slopes=1 to 1; $v=4.00$ feet per second; $h=0.249$ feet; W.S. at 100.000 feet. Flume elements: center depth 6 feet; $a=56.55$ square feet; $v=11.6$ feet per second; $h=2.092$ feet; $\Delta h=1.843$ feet. Total drop in $Z=1.1\ \Delta h=2.028$ feet]

Line	Item	Value of items at distance L from 0.00 of inlet transition						
		Canal	Transition					Flume
1	L, feet	0	2	4	6	8	10	12
2	Width T (scaled)	38.00	22.7	18.1	15.2	13.4	12.3	12.0
3	$q=656/T$		28.9	36.2	43.2	49.0	53.4	
4	d (taper from 5 feet to 6 feet)	5.00	5.17	5.33	5.50	5.67	5.83	6.00
5	$a=Td$ (preliminary)		117.4	96.5	83.6	76.0	71.8	56.55
6	a (after padding)		117.4	96.5	83.6	[1] 74.6	[1] 65.6	56.55
7	$v=656/a$ (6)	4.00	5.59	6.80	7.84	8.79	10.0	11.6
8	h	.249	.49	.72	.96	1.20	1.56	2.092
9	$H=d+h=(4)+(8)$	5.25	5.66	6.05	6.46	6.87	7.39	8.092
10	Z (as given)	100.000	—(1.1 Δh=2.028)=					97.972
11	$h_f=LS$ (assumed)	0	.02	.05	.07	.09	.14	.184
12	$E=E_o-(11)=Z_o+h_o-(11)$	100.249	100.23	100.20	100.18	100.16	100.11	100.065
13	$k=E-H=(12)-(9)$	95.000	94.57	94.15	93.72	93.29	92.72	91.973
14	Z (as developed) $=E-h$	100.000	99.74	99.48	99.22	98.96	98.55	97.972

[1] Between points 6 and 12 the area as in line 5 is "padded" with concrete to increase velocity in steady taper from point 6 to point 12. This padding likewise shapes the bottom from the rectangle at point 6 to the circle at the inlet of the flume proper.

For problems similar to this in which the total energy content H must be separated into its component parts, d and h, and which permit of using q instead of Q, as in the case of a rectangular prism, the curves given in figure 5 are offered. To illustrate: 50 second-feet per foot of width, with a total energy content, $H=6.70$ feet, will flow 3.28 feet deep at shooting stage, or 5.3 feet deep at streaming stage. Critical depth comes at $d_c=4.27$ feet, more or less. The difference between depth, d, and 6.70 is equal to the velocity-head, h.

Problem: Compute a warped inlet transition for our example flume, with canal and flume properties as listed at the head of table 6. Assume the water surface as following two identical parabolas; reversing curvature at half the length of the transition. Assume the inlet loss, including friction, to be $0.1\Delta h$, as in table 6. Thus the total fall in the water surface will be 2.028 feet. The first parabola will take half of this drop. For intermediate points the drop will be proportional to the square of the distance from the beginning. After the first curve has been developed, the same increments of drop are used, in reverse order, for the second curve. The essential computations are listed in Table 7. Complete data and explanations for such transitions were first developed by Julian Hinds (*14, p. 1437*). In this article Mr. Hinds points out that the hydraulics of the structure, excluding friction, are satisfied when the local areas have been obtained. He computes the local friction losses as separate items. The total length of transition assumed is slightly greater than that necessary to satisfy Mr. Hinds' suggestion of an angle of 12°30″ between the axis of the channel and the line joining the water edge in the canal to that in the flume at design-depth.

After computing the areas (line 5) any combination of depth and average width that will satisfy the local area, is possible. The writer chose the local depths (line 6) as steadily increasing from 5 feet in the canal to 6 feet in the flume. This yields average widths as given in line 7. The profile of the bottom and the shape of the average width curve were then platted and showed satisfactory curves. From the

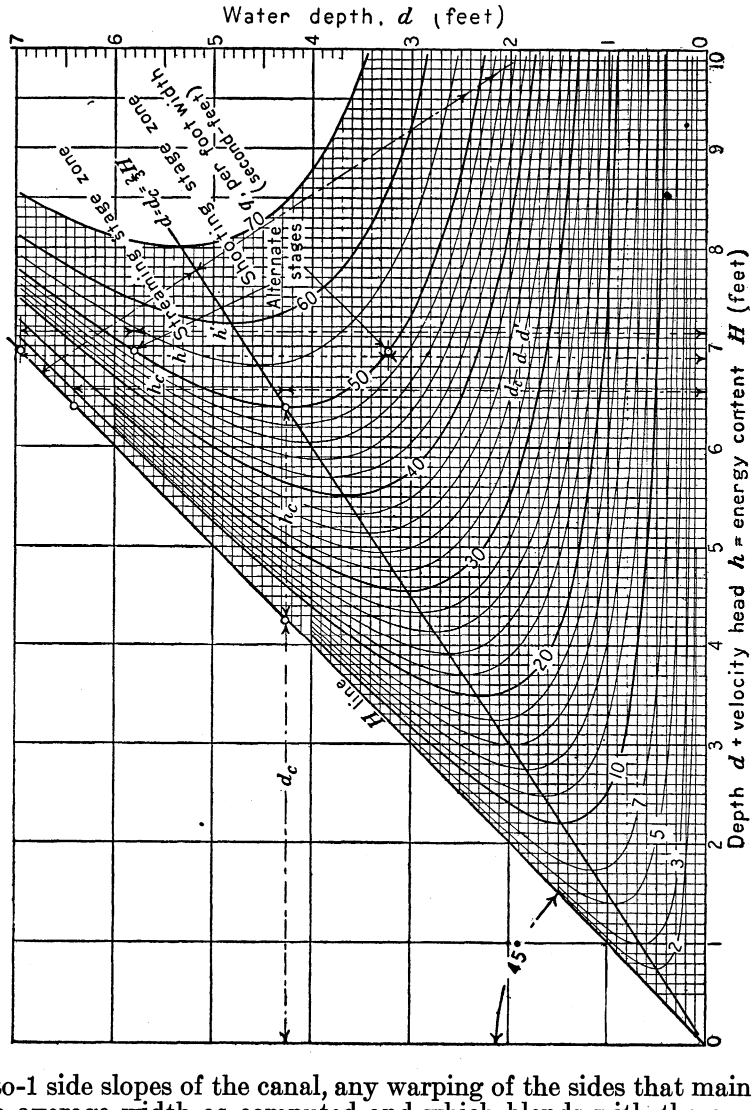

FIGURE 5.—Energy curves for flow per unit width in rectangular prisms.

1-to-1 side slopes of the canal, any warping of the sides that maintains the average width as computed and which blends with the curve of the flume entrance, will satisfy the problem. As the flume inlet is approached the warp may attain a vertical wall with the bottom rounded to conform to the projected elements of the flume proper.

TABLE 7.—*Computations for warped inlet with reversed parabolas for surface curves*

Line	Item	Stations								
		0.00	0+7.5	0+15	0+22.5	0+30	0+37.5	0+45	0+52.5	0+60
		Canal	Transition							Flume
1	ΔW.S.=drop in Z	---------	0.063	0.257	0.570	1.014	1.458	1.771	1.965	2.028
2	$\Delta h=(1)\div 1.1$	---------	.057	.234	.518	.922	1.325	1.610	1.786	1.844
3	$h=0.249+\Delta h$	---------	.306	.483	.767	1.171	1.574	1.859	2.035	2.093
4	v for h in (3)	4.00	4.43	5.57	7.02	8.68	10.06	10.94	11.44	11.60
5	Area, $a=Q\div v$	164.0	148.0	117.8	93.4	75.6	65.2	60.0	57.3	56.6
6	d, pro-rated	5.00	5.125	5.250	5.375	5.50	5.625	5.75	5.875	6.00
7	Av. width=(5)÷(6)	32.8	29.0	22.44	17.38	13.75	11.59	10.43	9.75	-------
8	Z=100.000−ΔW.S	100.000	99.937	99.743	99.430	98.986	98.542	98.229	98.035	97.972
9	$k=Z-d$	95.000	94.812	94.493	94.055	93.486	92.917	92.479	92.160	91.972

OUTLET TRANSITION

It is suggested that the reader review the comments on Outlet Losses on page 14. The outlet transition actually built for the flume in our problem (p. 67) is shown in plate 6, A. Here the velocity was reduced from nearly 11 feet per second to about 5.5 in the lined section of canal below. The type having flaring wing walls intersecting the line of the flume at 30° is common and quite satisfactory.

A recovery of 66 percent of Δh was accomplished in this structure. However, it is possible that a recovery of about 80 percent would have been made with a transition as developed by Hinds (*14, p. 1441.*)

Example: The data given and table 8 represent the results of computations for a metal flume outlet, modifying the outline by Mr. Hinds. The hydraulic properties assumed were as follows: A size 96 metal flume: $Q=50.8$ second-feet; $A=8.63$ square feet; diameter= 5.093; $d=2.24$; $V=5.89$; $h=0.539$. In canal, $Q=50.8$ second-feet; $b=5.0$ feet; $d=2.5$ feet; $V=2.32$ feet per second; $h=0.84$ foot; side slopes 1½ to 1.

The outlet transition is designed on the assumption that $0.8\Delta h=$ 0.364 foot will be recovered and but $0.2\Delta h$, including friction, will be lost. This design called for a structure of the same length as the one developed for the inlet transition. Since it is well known that flow may be accelerated more abruptly than it can be decelerated; wherever possible, the outlet transition should be longer than the inlet and especially where a maximum recovery of velocity head is needed.

TABLE 8.—*Computations for warped outlet transition, with reversed parabolas for surface curves*

Line	Item	Station						
		0+00	0+03.5	0+07	0+08.75	0+10.5	0+14.0	0+17.5
		Flume	Transition					Canal
1	ΔZ=rise in surface	0	0.030	0.018	0.182	0.246	0.334	0.364
2	$\Delta h=\Delta Z/0.8$	0	.038	.148	.228	.308	.418	.455
3	$h=0.539-\Delta h$	0.539	.501	.393	.311	.231	.121	.084
4	Mean velocity, v	5.89	5.68	5.03	4.47	3.86	2.79	2.32
5	Area= $Q/v=50.8/v=a$	8.63	8.94	10.10	11.36	13.16	18.21	21.90
6	0.5 b (assumed)	(1)	(1)	2.27	2.31	2.35	2.42	2.50
7	d (assumed)	2.24	2.24	2.24	2.33	2.34	2.46	2.50
8	Average width= a/d	--------	3.99	4.51	4.88	5.62	7.40	8.76
9	0.5 surface width=0.5 T	2.55	2.41	2.27	2.59	3.34	5.12	6.34
10	0.5 $T-0.5\,b$	(1)	(1)	0	0.28	0.99	2.70	3.84
11	Side slopes: 1	(1)	(1)	0	.12	.415	1.10	1.50
12	Z=142.03+(1)	142.03	142.06	142.15	142.21	142.28	142.36	142.39
13	k (not smooth)	139.79	139.82	139.91	139.88	139.94	139.90	139.89
14	k (revised)	139.79	139.82	139.87	139.89	139.89	139.89	139.89
15	Height of wall (top=143.39)	--------	3.57	3.52	3.50	3.50	3.50	3.50
16	(15)×(11)	--------	--------	--------	0.42	1.45	3.85	5.25
17	0.5 width of top of walls=(16)+(6)	2.55	2.41	2.27	2.73	3.80	6.27	7.75

[1] Pad outlet in first 7 feet from circle to rectangle with fillet radii of 2.55 at 0+00; of 2.08 at 0+03.50; and of 0 at 0+07.0.

Example: A critical-flow flume for maximum capacity in a short bypass around a dam under construction.

This example includes possibilities at critical, streaming, and shooting velocities. These conditions have been discussed extensively for some 15 years, but the conditions developing one or another of these flows are but little understood by many engineers otherwise generally familiar with flow formulas. They can be made clear in a single problem that arises frequently in flume design.

Problem: The most efficient rectangular box flume is required to bypass a river around a dam under construction: length of flume to be 500 feet; maximum flood anticipated around 1,000 second-feet; high-water line in the forebay formed by the upper cofferdam to be at elevation 100.00; the floor of the flume at the intake end to be at about elevation 94.00. Assume Kutter's $n=0.014$.

Since the water in the forebay is pooled (practically without velocity) the still-water surface is on the energy line, E. Therefore, at the upper end of the intake assume we have $100.00-94.00=6.00$ feet available for *energy content* $d+h$. Table 9 is set up to show the flows that would be discharged, per foot of intake width, for distributions of the energy content 6.00 feet into various combinations of $d+h$ that always total 6.00. If $d=0.00$, $h=6.00$ and the discharge would be 0.00 as there would result a high velocity but no depth and hence no area and no discharge. If $d=6.00$, again there would be no discharge since $h=0.00$ and hence $V=0.00$. However, for all distributions between these two there is both depth, d, and velocity head, h, so there must result some area of section, a, some velocity, v, and some flow quantity, Q. The figures in each column in table 9 result from the preceding data.

TABLE 9.—*Data for Q-curve, per unit width of rectangular channel, for energy content $H=d+h=6.00$ feet*

d	h	v	a	$Q=av$
0	6	19.64	0	0
1	5	17.93	1	17.93
2	4	16.04	2	32.08
3	3	13.89	3	41.7
3.5	2.50	12.68	3.50	44.4
3.9	2.10	11.62	3.90	45.3
$4.0=d_c$	$2.00=h_c$	$11.34=V_c$	4	$45.4=Q_c$
4.5	1.50	9.82	4.50	44.2
5	1	8.02	5	40.1
5.5	.50	5.67	5.50	31.2
5.9	.10	2.54	5.90	15
6	.00	0	6	0

The following argument will introduce the reader to the use of Q curves. Referring to figure 6, at the cross section under consideration—in the example at the flume intake—erect a vertical $E\ k$ on large-scale cross-section paper. For the various values of d as ordinates plat Q as abscissas. The resulting Q curve passes through a maximum at a depth of 4 feet. This is at critical depth, d_c. Note that it comes at two thirds H where H is the total energy content. Therefore the required flume intake to carry maximum flow has a capacity of 45.4 second-feet per foot of width, q, with 4 feet of energy content utilized in water depth and 2 feet in velocity head. The total width of intake would be $\frac{Q}{q}=\frac{1,000}{45.4}=22.0$ feet. In order that advantage may be taken of the maximum capacity through this intake at critical depth, the structure just beyond the intake must be able to carry the water at critical velocity or faster.

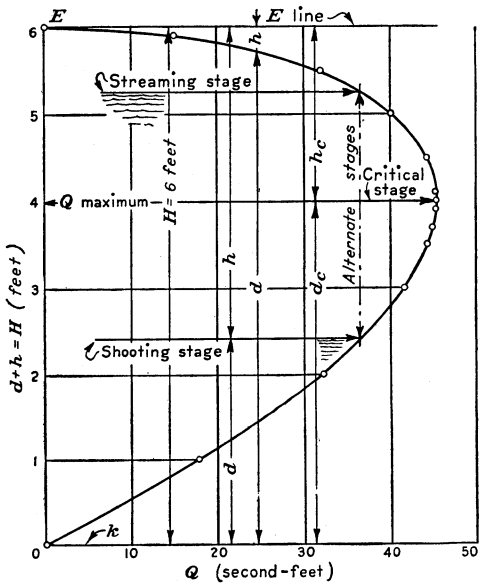

FIGURE 6.—The Q curve, showing the distribution between depth, d and velocity head h for any particular flow less than critical flow Q_c for any given value of H, which is the difference in vertical elevation between the energy line and the bottom of channel at any particular station. Such a curve can be developed for a whole channel as well as for unit flow. For still water W. S. and E are at the same elevation.

Table 10 has been prepared to show the uniform-flow possibilities of several flume sections that can be used: The given requirements are: $Q=1,000$; $n=$ (assumed) 0.014; $L=500$ feet; slope elastic for a short flume. Elements except slope are assumed for the necessary quantities and S is determined, by reference to figure 3, to satisfy the other elements.

TABLE 10.—*Trial set-ups for short flumes to carry 1,000 second-feet*

Flume symbol	Width	Depth	Area	Perimeter	Hydraulic radius	Slope	Velocity	Velocity head	Flow per foot of width	Critical velocity
	T	d	A	P	R	S	V	h	q	V_c
1	2	3	4	5	6	7	8	9	10	11
	Feet	*Feet*	*Sq. ft.*	*Feet*	*Feet*	*Feet per foot*	*Feet per second*	*Feet*	*Second-feet*	*Feet per second*
(a)	22	4	.88	30	2.93	0.0027	11.34	2.00	45.4	11.34
(b)	12	6	72	24	3.00	.0038	13.9	3.00	83.4	13.89
(c)	16	8	128	32	4.00	.00087	7.82	.951	62.5	16.04

If flume (*a*) at the same width as the intake be continued as a critical flow structure, a slope of 0.0027 would maintain a constant depth of 4 feet to the brink at the outlet end. To make final determination of the elevation of the flume bottom at the inlet the entry loss must be computed. This will be equal to $c_e\Delta h$ where $c_e=0.15$, including friction; therefore the total entry loss will be computed as $h_e=0.15\Delta h$, or $h_e=0.15\times 2.00=0.30$ foot. The total drop in water surface would be $(1+c_e)\Delta h=1.15\times 2.00=2.30$ feet. If the floor of the intake is set at 93.7, 2.3 feet is devoted to entry loss and the development of a velocity of 11.34 feet per second.

Flume (*b*) would require an additional drop, just below the intake end, sufficient to increase the velocity, V, from 11.34 to 13.9 or $\Delta h=3.00-2.00=1.00$ (column 9, table 10). This extra drop, plus friction loss, would be put into the upper end of the flume while the width is decreasing from 22 to 12 feet. Thereafter a slope of 0.0038 would maintain flow at critical depth (shown by conformity of columns 8 and 11) to the end of the structure.

Flume (*c*) would have a normal depth, d_n, of 8 feet, with normal velocity less than critical, so a definite control would take place at the outlet brink where critical depth would hold. Reference to figure 5 shows that critical depth in a rectangular channel, d_c, $=5\pm$ for $q=62.5$. Since this is much less than $d_n=8$ feet, there would be a drop-down curve extending upstream from d_c at the brink. Under this curve the depth is between d_c and d_n which is wholly in a zone of streaming velocities. However, it is seen that the water is to enter the flume at critical depth with a width of 22 feet and this width is to be narrowed to 16 feet. Use of figure 5 would show the rate of narrowing so that critical depth could be continually maintained, in theory, to the entrance of the 16-foot flume proper. The flow might then go through the hydraulic jump to approach d_n. Such a flume would be uncertain in its action and is not recommended.

It is well to reiterate that critical flow conditions can be used for short flumes but are not advisable for long ones. Having developed the maximum flow that can be discharged for the given stage in the pool, suppose the flume beyond the intake has such dimensions and slope that it can carry but 25 second-feet per foot of intake width. For the same pool stage, reference to the Q abscissa of the Q curve in figure 6 shows that 25 second-feet will flow through the intake at a depth of 5.7 feet with 0.3 foot invested in velocity head (again $5.7+0.3=6.00$ feet). Note that this point on the Q curve is above the apex. All such points indicate flow at the low-velocity or streaming stage. Directly below $Q=25$ for the streaming stage is $Q=25$ for a depth of 1.43 feet and a velocity head of 4.57 feet (again $d+h=H$ or $4.57+1.43=6$ feet). This flow would be at shooting stage. If the structure beyond the intake could carry 25 second-feet at the shooting stage, i.e., as a chute, the intake would not restrict its flow to 25 second-feet at the shooting stage but would build up a discharge to the maximum of 45.4 second-feet and perhaps overflow the walls of the flume at some lower point.

Problem. Determine the elements of a backwater curve with design Q', with a normal depth of 6 feet, checked so as to make a depth of 7.2 feet at the extreme lower end of the flume.

The detailed computations, given in table 11, are based on formula 18 (p. 17). The total change in depth from 7.2 feet back upstream to the normal depth of 6.0 feet is divided into several steps and the functions entering the Kutter formula, R, V, and S, are developed for the mean of the various steps, assuming a uniform flow through the particular step. As in problem 1, (p. 67), the basic elements for this problem include: $Q=656$ second-feet; $s=0.002$ foot per foot of length; and $n=0.012$. It is to be remembered that s is the bed slope, and S is the energy slope necessary to maintain flow for the mean elements as they are developed. The total length as given in the tabulations is always short of the actual length, the backwater curve becoming asymptote to the normal surface. Thus the total backwater curve can be taken as some 1,300 feet in length. The resulting curve is shown in the profile view in figure 8. Reference to Bakhmeteff (*4*) shows other curves that might be included on such a profile for special flow conditions.

TABLE 11.—*Backwater curve development for example flume described on page 67, from limiting depth of 7.2 at outlet back to normal depth of 6.0, with bed slope s=0.002 foot per foot*

Depth d	Area a	Hydraulic radius r	Velocity $Q/a=v$	Velocity head h	Energy content $d+h=H$	Average for reach L			Change in H ΔH	$s-S$	Local length $\frac{\Delta H}{s-S}=L$	Cumulative length ΣL
						r	v	S				
Feet	*Square feet*	*Feet*	*Feet per second*	*Feet*	*Feet*	*Feet*	*Feet per second*	*Foot per foot*	*Feet*		*Feet*	*Feet*
7.20	70.85	3.33	9.26	1.333	8.533							
						3.31	9.42	0.00118	0.107	0.00082	131	
7.00	68.50	3.29	9.58	1.426	8.426							131
						3.26	9.75	.00128	.095	.00072	132	
6.80	66.12	3.23	9.92	1.531	8.331							263
						3.205	10.11	.00141	.084	.00059	142	
6.60	63.74	3.18	10.29	1.648	8.247							405
						3.165	10.39	.00151	.037	.00049	75	
6.50	62.54	3.15	10.49	1.710	8.210							480
						3.135	10.59	.00159	.033	.00041	80	
6.40	61.36	3.12	10.69	1.777	8.177							560
						3.105	10.80	.00167	.027	.00033	82	
6.30	60.15	3.09	10.91	1.850	8.150							642
						3.075	11.02	.00176	.024	.00024	100	
6.20	58.95	3.06	11.13	1.928	8.128							742
						3.045	11.24	.00186	.021	.00014	150	
6.10	57.77	3.03	11.36	2.007	8.107							892
						3.015	11.48	.00196	.014	.00004	350	
6.00	56.55	3.00	11.60	2.093	8.093							1,242

The drop-down curve for the example flume (p. 67) from normal depth of 6 feet to its minimum under the condition of streaming flow, i.e., critical depth, is traced in table 12. Critical depth is taken from the intersection of two critical elements for a flow of 656 second-feet as graphed in figure 7. This occurs at 5.80 more or less. Since critical depth is but 0.2 less than normal depth, the drop-down curve is very short and the limit of a short flume for this condition would be about 200 feet. The computations are commenced at the brink where critical depth would hold. The resulting drop-down curve is shown in figure 8.

TABLE 12.—*Drop-down curve development for example flume described on page 67 from critical depth at 5.80± back upstream to normal depth of 6.00 feet, with a bed slope s=0.002 foot per foot*

Depth d	Area a	Hydraulic radius r	Velocity $Q/a=v$	Velocity head h	Energy content $d+h=H$	Average for reach L: r	Average for reach L: v	Average for reach L: S	Change in H ΔH	$S-s$	Local length $\frac{\Delta H}{S-s}=L$
Feet	*Sq. ft.*	*Feet*	*Feet per sec.*	*Feet*	*Feet*	*Feet*	*Feet per sec.*	*Ft. per foot*	*Feet*		
5.80	54.15	2.94	12.11	2.284	8.082						
						2.944	12.05	0.002221	0.001	0.000221	5
5.85	54.74	2.95	11.98	2.233	8.083						
						2.960	11.92	.002160	.002	.000160	13
5.90	55.32	2.97	11.86	2.185	8.085						
						2.976	11.79	.002099	.002	.000099	20
5.95	55.95	2.98	11.72	2.137	8.087						
						2.990	11.66	.002044	.006	.000044	136
6.00	56.50	3.00	11.60	2.093	8.093						
						Total L					174

Problem. To determine the conformity between computed and measured locations on a drop-down curve, or to determine the friction factor n with field data secured where a definite drop-down curve or backwater curve exists. In the capacity test of Agua Fria flume, sections were developed and water-surface elevations taken at intervals of about 500 feet throughout the length of nearly 6,000 feet. When platted, it was found that quite uniform flow extended for the reach of 2,160 feet used to determine the friction factors. (See no. 104, table 2.) For some 3,000 feet at the lower end of the flume the freeboard increased continually, indicating the development of the drop-down curve. From the field data the computations shown in table 13 were developed. Starting at the water surface in the outlet end of the metal flume, the drop-down curve was computed to determine the distance back up the flume to the next location where surface and sectional elements had been measured. For determining the values of S, the friction factor $n=0.0137$, determined in test no. 104, was used. In the last column is shown the distance as measured on the ground (the distance between stations as listed in column 1). Considering the high velocities and consequent roughness of the water surface, the writer considers the conformity acceptable.

TABLE 13.—*Elements for drop-down curve at the lower end of Agua Fria flume, no. 104. Shows conformity between computations and field measurements made at the stations as listed* [1]

Station	Elements (see notation)							Cumulative lengths	
	r	v	h	Z	E	ΔE	S	Computed ΣL	Measured ΣL
	Feet	*Ft. per ft.*	*Feet*	*Feet*	*Feet*	*Feet*	*Ft. per ft.*	*Feet*	*Feet*
59+58		8.15	1.033	13.11	14.143				
	2.54	7.47				0.732	0.00114	642.0	596.0
53+62		6.78	.715	14.16	14.875				
	2.74	6.52				.652	.00112	1,224.0	1,314.0
46+44		6.25	.607	14.92	15.527				
	2.87	6.00				.637	.00066	2,190.0	2,036.0
39+22		5.75	.514	15.65	16.164				
	2.90	5.75				.407	.00060	2,868.0	2,738.0
32+20		5.76	.516	16.055	16.571				

[1] The slope, S, is necessary to maintain flow for the average velocity and for the average value of the hydraulic radius, r, over the distance between stations as measured, with Kutter's $n=0.0137$, as determined for the reach of uniform flow upstream from station 32+20. (See no. 104, table 2.) From observed elevations of water surface and elements of cross-section developing velocity, and hence velocity head, the elevations of E $(=Z+h)$ were computed. From these values of E the items ΔE were computed. The length of channel necessary to develop a given difference in E is found by dividing ΔE by S. Computations are by slide rule.

If it is desired to compute a value of n for such a drop-down curve then the data for all but the two columns for S and computed L can be listed. Items for values of S can be computed from $\frac{\Delta E}{L}$ where L is the measured length between stations under consideration (not the cumulative length). From the various values of S thus developed the values of n can be taken from figure 3. The average value of n will be quite close to that resulting from test of a reach approximating normal flow conditions.

CHARACTERISTIC CURVES

The flow-behavior of an important flume structure, either contemplated or already constructed, can now be studied by means of a set of curves showing the conditions of flow throughout great variation in many of the essential elements. Some of these curves have been shown in various publications but others are offered for the first time, so far as the writer is aware.[15] They are analogous to the familiar characteristic curves for pumps.

A single flume (no. 301, et seq.) will be used as the example for the curves as it was for the solution of ordinary normal-flow problems (p. 67). In commercial operation, this flume shows some of the unusual characteristics brought out by the curves, and also shows changes in these characteristics from one year to another. Hence it can be used to advantage in studying the characteristics of other flumes. It is not suggested that similar studies be made in this comprehensive manner regarding all flumes, but the characteristics of a costly structure, designed to carry a full load of water at a velocity approaching the critical, in a locality where the flume interior may change appreciably during the irrigation season, may be so graphed

[15] The solution of one or more related subproblems generally leads to obtaining direct answers to most engineering problems, but most problems involving the flow of water in open channels, on the other hand, are not solved by one direct set of solutions, but by successive trials, gradually approaching an acceptable result. This condition makes a set of curves of particular value. Trial answers, approximately correct, can be quickly obtained. When the most acceptable answer is found, actual computations can be made, if desired, for formal reports, court testimony, or other use.

that its performance can be anticipated for all changes of conditions that are liable to occur. Furthermore, the curves will show the improvement in capacity that may result from proposed maintenance work that would change one or more of the elements affecting capacity curves.

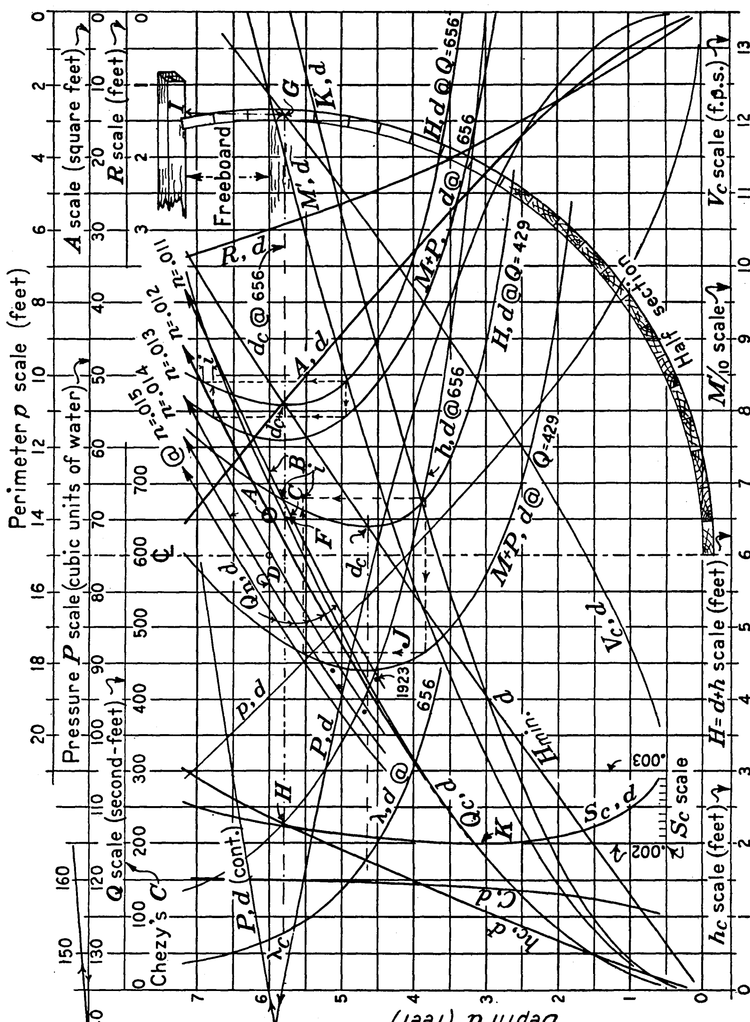

FIGURE 7.—Characteristic curves for semicircular flume, 12 feet in diameter. For explanation see p. 79 et seq.

The curves are divided into class A—characteristic curves dependent only on shape and dimensions of water prism, and class B—characteristic curves, dependent on curves in class A, and also dependent on other functions, such as slope, S, and value of retardation coefficient, n, and in addition perhaps dependent on a given quantity of flow, Q.

The curves are first listed. The uses for which each one is helpful follow the listing.

The flume referred to is constructed of wood staves and is supported in cradles so that the water prism occupies the lower segment of a circle 12 feet in diameter. Computations for normal flow capacity were based on a segment up to mid-diameter. Above that elevation there is a freeboard of about 1.2 feet which may be encroached upon to a moderate extent. The computed curves are extended to include depths up to the crossbar.

The characteristic curves are shown in figure 7. The tables upon which they are based were developed in detail and plotted on cross-section paper, spaced 10–10, to the inch. To save space only a few values of d were tabulated for each element given in the example tables that follow. Many of the curves are dependent on other curves. The order of development used gives various items as they are needed (see tables 14 and 15).

TABLE 14.—*Abstracted elements for drafting of characteristic curves dependent only on shape and dimensions of water cross section*

Ratio, depth to diameter	Depth of water prism	Area of water prism	Wetted perimeter	Hydraulic radius	Width, water surface	Velocity head for V_c	Critical velocity for h_c	Critical flow $Q_c = AV_c$	Minimum energy content $= d_c + h_c$	Bakhmeteff's $M' = A\sqrt{A/T}$	Static pressure in water prism
d/D	d	A	p	R	T	h_c	V_c	Q_c	$H_{min.}$	M'	P
1	2	3	4	5	6	7	8	9	10	11	12
	Feet	*Sq. ft.*	*Feet*	*Feet*	*Feet*	*Feet*	*Ft. per sec.*	*Sec.-ft.*	*Feet*		*Cu. ft.*
0.10	1.2	5.9	7.8	0.76	7.2	0.41	5.1	30	1.62	5.3	2.9
.25	3.0	22.1	12.6	1.76	10.4	1.06	8.3	183	4.06	32.3	27.1
.40	4.8	42.3	16.4	2.57	11.7	1.80	10.8	455	6.60	80.2	84.8
.50	6.0	56.6	18.9	3.00	12.0	2.36	12.3	696	8.36	122.8	143.9
.60	7.2	70.8	21.3	3.33	11.7	3.02	13.9	987	10.22	174.1	220.5

TABLE 15.—*Abstracted elements for drafting of characteristic curves dependent on shape and dimensions of water cross section and also on one or more of the elements of friction coefficient, slope and quantity of flow*

[This table is supplementary to table 14, from which column 2 is repeated]

	Dependent on S=0.002 and n=0.012				Dependent on Q=656 second-feet					
	Chezy's $C = \frac{V}{\sqrt{RS}}$	Bakhmeteff's K, $=AC\sqrt{R}$	Critical slope, $S_c = g\frac{(M')^2}{K^2}$	Normal $Q_n = K\sqrt{S} = K\sqrt{s}$	Mean velocity, V, $=Q/A$	Velocity head	Energy content, H $=d+h$	Momentum, M $=Q^2/Ag$	Momentum+pressure= $Q^2/Ag+P$ (see column 12, table 14)	Kineticity factor, $\lambda=2h/d$
d	C	K	S_c	Q_n	V	h	H	M	$M+P$	λ
2	13	14	15	16	17	18	19	20	21	22
Feet			*Ft. per foot*	*Sec. feet*	*Ft. per sec.*	*Feet*	*Feet*		*Cu. feet of water*	
1.2	121.4	624	0.00234	27.9	111.4	--------	--------	2,272.0	2,275.0	--------
3.0	139.3	4,084	.00201	182.6	29.7	13.7	16.7	605.2	632.3	9.13
4.8	146.5	9,920	.00210	443.7	15.52	3.76	8.56	316.5	401.3	1.57
6.0	149.3	14,600	.00227	653.3	11.60	2.09	8.09	236.6	380.5	.70
7.2	151.2	19,550	.00255	874.5	9.25	1.33	8.53	188.9	409.4	.37

CLASS A CHARACTERISTIC CURVES

1. The half-section curve shows a cross section of one half of the symmetrical flume (fig. 7). The shell was made of 46 identical staves. Half of these are shown. A horizontal line from centerline to shell gives $T/2$ for any given d. (See columns 2 and 6, table 14.) The stave-marks give gage readings for any field observations of the surface curve when taken in terms of stave freeboard.

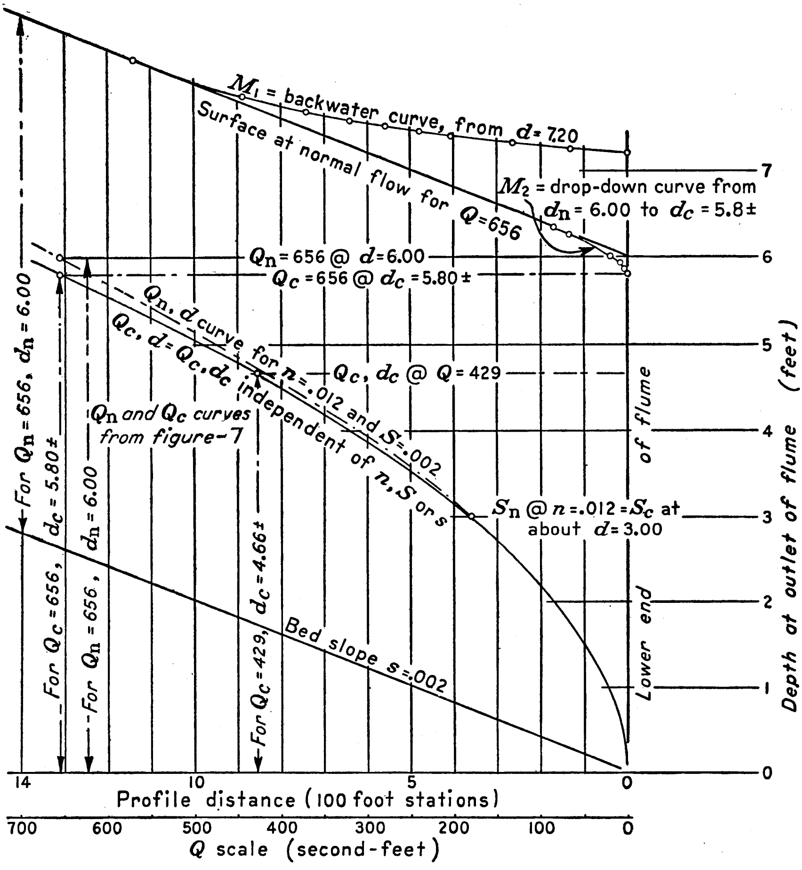

FIGURE 8.—Backwater and drop-down curves in profile of flume used as example for characteristic curves shown in figure 7.

2. The A,d curve shows area, A, of the water section for any given depth, d. Based on columns 2 and 3, table 14.

3. The p,d curve shows wetted perimeter, p, for any given depth, d. Based on columns 2 and 4, table 14.

4. The R,d curve shows hydraulic radius, R, for any given depth, d. $R=A/p$. Based on columns 2 and 5, table 14.

5. The h_c,d curve shows the velocity head for the critical velocity at any depth, d. By formula $h_c=A/2T$ for any shape of section. Based on columns 2 and 7, table 14.

6. The V_c,d curve shows the critical velocity, V_c, for any depth, d. If the water is flowing at critical velocity it is also at critical

depth. Under this condition the given value of $d=d_c$. Based on columns 2 and 8, table 14. $V_c=\sqrt{g}\ \sqrt{A/T}$

7. The Q_c,d curve shows the quantity, Q, flowing at critical velocity, V_c, as taken from curve 6 above, at a value of d which again becomes d_c, as for curve 6. $Q_c=M'\sqrt{g}$. Based on columns 2 and 9, table 14.

8. The $H_{\text{min.}},d$ curve shows minimum energy content, $H_{\text{min.}}$, where $H=d_c+h_c$ for any value of d as d_c. Based on columns 2 and 10, table 14. Note that this curve intersects the energy curves for $Q=429$ and for $Q=656$ at their respective critical depths. Since any energy curve is rather flat around the apex, the critical point is uncertain when taken from a graph. This intersection removes this uncertainty.

9. The M',d curve shows Bakhmeteff's M-function for any value of d, $M'=A\sqrt{A/T}$. Based on columns 2 and 11, table 14.

For every flow Q there is one depth of water in a channel that is critical. For some shapes the direct solution is simple but not for others. With characteristic curves developed, this depth can be taken directly from Bakhmeteff's M-function curve. For a given Q the value of $M'_c=Q/\sqrt{g}$.

On the M',d curve, for the above value of M'_c, the corresponding value of d will be critical for the given Q.

10. The P,d curve shows the pressure on cross section of water prism, in cubic units of water, for any given depth, d. Values from this curve, combined with kinetic units for any given flow, Q, give the elements for a momentum curve for a particular flow. See column 21, table 15. Curve 10 based on columns 2 and 12, table 14.

CLASS B CHARACTERISTIC CURVES

11. C,d curve, showing values of Chezy's C for normal flow at any depth, d. In this particular case, based on a value $s=0.002$ and Kutter's $n=0.012$. This curve slightly different for different values of s up to $s=0.001$. when based on Manning's formula, the values of C are the same for all reasonable values of S but vary with value of n chosen. Tabular values from columns 2 and 13, table 15. $C=\dfrac{V}{\sqrt{RS}}=\dfrac{V}{\sqrt{Rs}}$ for normal flow.

12. The K,d curve, showing Bakhmeteff's conveyance factor, K, for any value of d, based on values of Chezy's C from the C,d curve. Thus K is a characteristic for our particular channel for values of $n=0.012$ and $s=0.002$ and is sufficiently close for values of s down to 0.001. The platted values based on columns 2 and 14, table 15. $K=AC\sqrt{R}$. If K is based on the Kutter formula it varies slightly for slopes flatter than 0.001.

13. The S_c,d_c curve showing critical slope, S_c, necessary to maintain uniform flow at critical depth, d_c. In the example, this slope agrees with that used in construction (0.002) for a depth of about 3 feet and is sufficiently in conformity at any depth so that the effective energy slope might equal or exceed the critical slope for short reaches of flume or for reaches slightly smoother than the average. $S_c=g\dfrac{(M')^2}{K^2}$. Based on columns 2 and 15, table 15

14. The Q_n,d curve shows the quantity, Q, that will flow at a theoretically uniform rate for any value of d with a value of $n=0.012$ and for a bed slope of $s=0.002$. This condition makes $d=d_n$; greater than d_c for streaming, tranquil flow; equal to d_c for glassy, critical flow; and less than d_c for shooting, torrential flow. Since the flow is assumed to be uniform, in a uniform-shaped channel, the slope of the bed is parallel to that of the water surface and of the energy gradient. Based on columns 2 and 16, table 15. Note that column 16 is based on the conveyance factor, K, and shows the reason for this characteristic name. Bakhmeteff calls this the "normal discharge curve." $Q_n=K\sqrt{s}=K\sqrt{S}$ since $s=S$ for normal flow.

The upper portions of similar curves based on the same slope but on values of n from 0.011 to 0.015 are also given. The use for these will be explained later.

The rest of the curves in this table are based on a given flow—in this case design Q of 656 second-feet. Curves for V and h are not plotted. The data are listed in columns 17 and 18, table 15. The data are necessary only to develop column 19 from which comes:

15. The H, d curve, usually called the "energy curve", showing the energy content, $d+h=H$ for any given depth d but for a particular value of Q. Based on columns 2 and 19, table 15. This curve is independent of s and n but is different for each value of Q. It is valuable for tracing the surface curve through any open constriction in a canal, such as an inlet transition followed by a short flume and an outlet transition. For the same value of H, the opposite point on the curve shows the alternate stage at which the given Q might flow with the same energy content. Assuming shooting flow and the hydraulic jump could occur in the flume without loss, then the upper stage or depth can be picked off directly from this curve. However, there is always a loss in the jump; therefore, this curve must be taken in conjunction with no. 16 below to show the stage, including loss, after the jump.

16. The $(M+P)$, d curve, usually called the momentum curve, shows the momentum, M, plus pressure, P, in cubic units of water for any given depth for a given flow, Q. Based on columns 2 and 21, table 15. It is a companion curve to the energy curve, no. 15 above, for the same Q, and is valuable in showing the height of jump that may be expected where conditions develop the jump in a flume.

17. The λ, d curve, showing Bakhmeteff's kinetic flow factor, λ for any depth, d. Based on $Q=656$ second-feet. This curve is characteristic of our especial channel with a particular flow, but is independent of s and n. The platted values are based on columns 2 and 22, table 15.

USES OF THE CHARACTERISTIC CURVES

The many uses for design, maintenance, and operation that can be obtained at a glance or with a minimum of computation from the curves shown on figure 7 are explained in the following paragraphs. Specific points referred to below are indicated in the figure by the same letters:

At point A is found the design Q' of 656 second-feet, using as the values of n, 0.012; S, 0.002; and R, 3.00 (corresponding to $d_n=6.0$). Corresponding to $Q'=656$ the value of $V=11.6$. These are the four elements that usually constitute the basis of computing the capacity or hydraulic performance of a flume, aside from entry and exit phenomena.

Running through the point A is the Q_n, d curve for a value of $n=0.012$. Reference to the Q-scale shows the quantity that will be in the flume for any given depth or vice versa, so long as the value of n remains at 0.012. If the surface when new is better than anticipated and the value of n is 0.011, then the point B indicates that about 717 second-feet can be carried at design depth of 6 feet; or point C indicates that design Q will flow at a depth of but 5.7 feet. On the other hand, if algal growth causes the value of n to become 0.014, the flume will carry but 562 second-feet at 6 feet depth (point D), and the design Q of 656 second-feet can only be carried with a normal depth of 6.55 feet, reducing the theoretical freeboard from 1.2 feet to some 0.65 foot.

Between Q_n, d for $n=0.012$ and for $n=0.011$ is found the Q_c, d curve. Note that this curve merges with Q_n, d for $n=0.012$ at $d=3.0$ feet. Point F shows that design Q' of 656 will run at critical depth if n is just a little lower than 0.012. Passing horizontally from F to G, it is found that the velocity for such a critical flow would be nearly 12.1 feet per second. This is a very high velocity for any channel of less incline than that of a chute drop. Returning horizontally to the left from G to H, the velocity head for this velocity is about 2.3 feet, which exceeds the freeboard from G to I by about 0.9 foot. If one could be assured that the flow would hold at critical depth, this excess might not show danger of overflowing because flow just at the critical is a smooth, often perfectly glassy, flow. However, at the energy curve for $Q=656$ there is a very flat curve in the region of the critical depth, indicating that a range of depth, say from 5.4 feet at shooting flow to 6.4 feet at streaming flow, can take place with practically the same energy content, i.e., with the same elevation of the energy line above the bottom of the flume. This condition indicates that the water surface throughout the length of the flume would be liable to a wide range of depth, depending on minor effects of flume surface and curvature. It also indicates that retardation of four reverse curves (pl. 13, A), even of 500 feet radius, may throw the flow through the hydraulic jump, and that the roughness of the jump wave may overtop the flume. On the other hand, a slight algal growth that would probably exist by the time the full flow of the flume is required would narrow the range to the extreme flat apex of the energy curve and change of flow from shooting to streaming stage might take place without an appreciable jump. In June 1923 when this flume was new, the value of n was actually 0.0112, and normal flow was faster than the critical, as can be seen by the platted point (1923) on figure 7, corresponding to elements in no. 302.

Again, referring to point F, corresponding to $Q=656$ and critical depth for that particular Q, this critical depth, d_c, is about 5.8 feet above the bottom. This has particular significance in connection with the water stage in the canal below the flume outlet. So long as this stage is at an elevation equal to or below that of the water surface in the end of the flume at $d_c=5.8$, plus any recovered velocity head h_r at the flume outlet, the stage at the flume end will not go below critical depth of 5.8 feet with the exception mentioned below. This is true because the discharge is already a maximum for any slope and value of n that will generate a normal velocity at or slower than critical. For all lower stages of water in the canal below, water would pour over the brink at the flume outlet at a depth of 5.8 feet.[16]

The exception to this condition occurs when the value of n, say for the new flume, was less than 0.0113 which corresponds to $d_n = d_c = 5.8$ for $Q = 656$ second-feet. For values of n less than 0.0113, d_n is less than d_c and there would be no material drop-down curve toward the flume outlet, the jet shooting out at the end with approximately normal depth.

Assume that it is proposed to increase the capacity of the flume by lining the canal below the flume: The lining would lower the stage for any flow and appear to provide a steeper gradient in the flume above, thus increasing the velocity and the maximum capacity without encroaching on the former freeboard.

This process would be successful for any length of flume less than the length of the drop-down M_2 curve (fig. 8), in the particular flume only about 200 feet. In other words, the drop-down curve extends about 200 feet from a normal depth of 6 feet to the critical depth of 5.8, and any additional lowering of the stage below would not change the length of this curve. Furthermore, the improvement by lining the canal would effect only this length of 200 feet and the upper end of the flume would serve as the criterion for capacity, its maximum capacity still being the capacity of the structure. The writer knows of several locations where the capacities of long flumes required improvement and this method was tried. In all cases the drop-down curve intersected the original surface some distance above the outlet and from that point upstream the conditions were as before. In the argot of the ditch rider, a "good-get-away" (i.e., critical flow at an outlet) will improve the capacity of a short flume but will not help a long one.

Likewise, a high stage in the canal below a flume outlet will lower the capacity of all flumes—essentially by raising the stage at the lower end of the flume and encroaching upon freeboard. In figure 8 the backwater M_1 curve is calculated for a stage at the flume outlet of $d = 7.2$ feet instead of the design depth of 6 feet. For any stage at the outlet between 6 and 7.2 feet in depth, the corresponding portion of the backwater curve may be used. In such a curve it must be remembered that velocities are becoming less as the depth increases and therefore the velocity-head requirement also becomes less. Practically all of the difference in velocity heads is recoverable in a perfect transition section such as a wedge-shaped flow in a uniform flume channel. This recovered velocity head is available to overcome friction or to raise the surface of the water in the flume. In actual operation the water surface toward the end of such a checked flow has but little or no slope, the energy to overcome friction being provided largely by the recovered velocity head.

From the M_1 curve and the accompanying table 11 it appears that backwater effect extends some 1,300 feet up the flume. Above that point normal flow is approximated. In this particular case the flume,

[16] Critical depth comes exactly at a brink for a very small flow. For all greater flows critical depth comes a short distance upstream from the brink. However, this distance is so short that it is ignored in most hydraulic problems. It cannot be neglected if the use of the brink as a critical-flow meter is contemplated. Here the difficulty lies in determining just how far above the brink critical depth can be found for any given flow.

if less than 1,300 feet long, would be classed as a short flume under the definition given on page 8.

When study of the characteristic curves begins to indicate that critical flow conditions are likely to be encountered, the S_c,d curve should certainly be platted. Examination of this curve on figure 7 shows that the effective critical slope equals 0.002 for a depth of about 3 feet at point K. This is also brought out by the intersection of the Q_n,d (for a value of $n=0.012$) curve and the Q_c,d curve coming at a value of $d=3.0$ feet. For a full load of 656 second-feet at normal depth, the critical slope S_c is found to be 0.00227, which does not mean that the bottom would have to take this slope, but if the slope of the energy gradient becomes as steep as 0.00227 then critical flow may develop. If this takes place, then flow faster than the critical nearly always follows, except at a brink. If the flow becomes faster than the critical, then the jump will probably result.

Likewise for the critical depth corresponding to a flow of 656 second-feet $S_c=0.00223$ which is designated by Bakhmeteff as the normal critical slope.

The momentum $(M+P)$, d curves and the energy curves for design-Q′ of 656 and also for observed $Q=429$ are drawn. Opposite points on the energy curve show the two stages on the assumption that there is no loss in the jump. However, the momentum theory holds true when the loss is included, therefore the corresponding opposite points on the momentum curves show the true heights of the jump.

On the curves for $Q=429$, assume that a jump takes place from a shooting-flow depth of 3.83 feet. From this depth on the H, d (energy) curve, follow the dashed example to intersection with the momentum curve, thence upward to the alternate stage of about 5.53, thence back to the energy curve. Note that the last intersection is below the stage opposite the point of beginning on the energy curve. This upper stage would occur at about 5.85 feet. The difference between 5.85 and 5.53, or 0.32 feet, indicated by i in figure 7, gives the loss that might be expected through the jump under the conditions assumed for the example.

Between the two curves for $Q=656$ second-feet a similar dashed example shows that there is much less loss in a jump where the energy and momentum curves are nearly parallel; that is, for relatively deep water prisms. If the loss of head is relatively small, the recovery of head in that particular jump is relatively great.

Hinds (*13*) has pointed out that if the momentum curves for the shooting stage before the jump and for the streaming stage after the jump be platted on the profile of the structure the two curves will intersect at the location of the jump. A mere touching tangentially would indicate that flow would pass from the shooting stage to the streaming stage without the jump.

The λ, d curve for any particular Q shows the relative dominance of kinetic effect as flow passes through critical stage to shooting stage. In the special problems shown by Bakhmeteff (*4*), this function is used extensively, especially in connection with problems of the hydraulic jump.

Problem. After designing the example flume for 656 second-feet, how can the design capacity be increased to carry 700 second-feet?

This can be accomplished in several ways: By increasing the slope; by encroaching on the freeboard as originally set up; or by securing and maintaining a very smooth surface.

(1) By change of slope, using same depth, $d=6.0$ and value of $n=0.012$ as before. By slide rule:

$$\text{The new slope, } s, =\frac{Q^2}{K^2}=\frac{700^2}{14{,}650^2}=\frac{490{,}000}{214{,}500{,}000}=\frac{49}{21{,}450}=0.002284.$$

The value K is taken directly from the K,d curve.

(2) By encroaching on freeboard but keeping the same slope and value of n. On the Q_n,d curve for $n=0.012$, $Q=700$ on a depth line of 6.23 which can be accepted as satisfactory for emergency peak loads as long as the value of n remains at 0.012 or less. If, however, algae increase the value of n to 0.015, $Q=700$ on the depth line of 7.1, which would use practically all of the freeboard, making certain the overtopping of the flume sides with resulting erosion at flume supports.

(3) By a very smooth surface. The intersection of $Q=700$ and $d=6.00$ comes on a Q_n curve for a value of Kutter's n of about 0.0113, which also happens to be on the Q_c curve for any value of n. This would indicate a flow at critical depth with a value of n that could be expected for a new stave flume, but which would be difficult to maintain. Combining the figures of (2) and (3), the deduction is reached that a flow of 700 second-feet would fill the flume to a depth approximating 6 feet, at critical flow, when the flume was new and would gradually encroach upon freeboard until it required a depth of more than 7 feet with $n=0.015$.

Problem. Determine the normal depth d_n for a flume similar to the example on page 67 at $n=0.012$ for $Q=400$, at a slope of 0.001. Note that the Q_n curves shown on figure 7 were for a slope of 0.002. By slide rule:

$$K=\frac{Q}{\sqrt{s}}=\frac{400}{0.0316}=12{,}660.$$

On the K,d curve $K=12{,}660$ at a depth of 5.5+ feet. For slopes flatter than 0.001 the values of Chezy's C change when considered in terms of the Kutter formula. If the problem had assumed a slope of say 0.008 it would perhaps be advisable to compute and draft a new K,d curve.

Problem. In economic problems regarding conveyance of water for hydroelectric projects and those involving pumping lifts it may be desirable to determine the power required to overcome friction. Determine the power in horsepower per mile of flume for the standard flume example ($Q=656$; $s=0.002$; $n=0.012$; $R=3.0$). Bakhmeteff (*4, p. 21*) shows that $\frac{62.4Q^3}{550K^2}=$ the horsepower lost per foot. By slide rule, $\frac{62.4\times 656^3\times 5{,}280}{550\times 14{,}650^2}=788$ horsepower per mile.

Bakhmeteff (*4*) describes other functional symbols that aid in the development of backwater and drop-down curves for the rectangular and other specified shapes. Some of them present difficulties for circular or catenary shapes. He also shows "delivery curves" for all stages of head and tail waters for short channels (flumes) between two reservoirs, and other special problems that do not often occur in irrigation practice.

CHUTE FLUMES

In the previous discussion, the flume has been considered in its usual state of flow; a moderate slope, generating a velocity slower than the critical as a rule, but a high velocity when compared with ordinary flow in open earth channels. In exceptional cases, such as the example flume (no. 301), the normal flow for any quantity may be designed for a velocity slower than the critical but the initial surface is better than anticipated and the velocity exceeds the critical, at least during the first few months of operation. This flume was not designed as a chute. (See p. 67.)

A chute flume is used to lower water from one elevation to another down a sharp incline. It is an inclined drop structure. In the Northwest there are many chutes conveying water from canals on bench land to similar canals or laterals on the bottom land (pl. 11, D). Stockton (*26*) mentions some 74,000 feet of timber and metal flume chutes on the western section of the Canadian Pacific Railway project, near Calgary, Alberta. The terrain of the Northwest, shaped by action of the ice sheet of the last glacial period, requires many ordinary flume and chute structures to convey water to the desired locations. The chute flume is extensively used to convey water from a feed canal to a reservoir at any stage of the latter. Sometimes the flume is extended down the steep reservoir side to a point near the reservoir bottom—at least to a point where water can be released on the reservoir floor without causing serious erosion. Sometimes the chute is extended to a point just above the high-water line of the reservoir and the jet thrown as far as possible from the foundation of the end of the chute (pl. 11, A, B). Chutes convey water from crest spillways of reservoirs to some point where the water can be released down the cliffs without objectionable erosion or the chute may continue to discharge the water at river level below the dam. The side-channel spillway usually feeds a chute.

In hydroelectric construction, the chute is often used to convey rejected water from a surge chamber to some nearby stream channel.

In both irrigation and hydroelectric practice, the chute flume is used to convey excess water from a waste gate set in the side of the main conduit. Exceptional topography and hard-rock terrain may allow waste water to be thrown directly from the side of the conduit. The criterion for all these chute structures is that the flume shall be long enough to insure that the released water will be powerless to injure the works by erosion.

The flow of water in a chute is essentially different from that in a flume of ordinary slope.

(1) Velocities encountered are several times those found in ordinary channels and usually far above those most common in ordinary flumes. Velocities of from 10 to 15 feet per second are relatively slow, from 15 to 30 feet are usual, and exceptional velocities run to 40 or 50 feet while one chute in Europe is reported to flow at some 80 feet per second.

(2) Obviously such velocities are all faster than the critical. The kinetic effect far overshadows the static pressures in the water prism.

(3) The sequence of flow conditions in a flume chute usually is as follows: With water delivered to the head of chute at a velocity below the critical.

If the head of the chute is a crest, weir flow comes over the crest at critical depth. If the head begins at a channel, critical depth is attained just above the brink where the sharp incline begins. From the point of critical depth, velocity is at shooting stage and is increased until normal flow is approximated or until the chute ends. As in the case of a short flume, the structure may end before normal flow is established.

However, if the approach to the chute is not in the same direct line or if the approach is wide and the chute is sharply narrowed near the top, uncertain flow will result. The resulting water surface does not take the smooth, glassy characteristics of critical flow, but starts with a series of high rough waves capped with white water. Beyond these waves the flow often slashes from side to side, showing little conformity between design and results.

(4) At the lower end of the chute, many conditions may hold. Water may leave the chute to enter a canal that will flow at streaming stage on an ordinary slope. At the lower end of the chute, the water may drop over a brink into a pool, or the jet from the chute may enter the pool from one side or end. In either case the extreme kinetic effects must be materially reduced by means of energy dissipators.

(5) Where the water surface at the end of the chute is above the surface of the pool, the energy dissipator is usually a confined water cushion or some form of labyrinth, or the jet is divided and turned to impinge upon itself.

(6) If the jet enters the pool at about its own level, the principle of the hydraulic jump may be utilized (pl. 11, D). This is an effective dissipator if the jet is made wide and thin before entering the pool. Stevens has pointed out the percentage of energy that may be dissipated under various conditions (*25*).

(7) Where a flume chute feeds a reservoir, conveying the water down the inclined bank until the reservoir surface is reached, whatever the stage of the water, there is no opportunity or necessity of widening the jet and the jump occurs as the swift water impinges upon the still water of the reservoir (pl. 11, C). A full reservoir may back the water into the canal feeding the chute so that no jump is encountered as shooting flow does not develop (*4, p. 58*).

(8) Water flowing swiftly down a long flume chute does not follow the law of hydraulics usually taught as infallible; the continuity equation $Q=AV$ for any value of A whatsoever. This exception to the rule is due to the entrainment of air, gradually creeping into the jet from the sides until finally the whole prism is composed of white water. In a report of the chutes on the Boise project of the United States Bureau of Reclamation, Steward[17] showed that this entrainment may expand the water prism by 30 percent or more. The quantity, Q, was measured by current meter or weir in the canal above or below the chute. The cross sections of the white water (water plus entrained air) were taken as usual, from which the values of r were computed. The actual mean velocity of the water down the chute was determined by timing color down the incline.

[17] STEWARD, W. G., Op. cit. (see footnote 11).

It was found that this measured V exceeded the velocity as computed by the equation $V=\frac{Q}{A}$ by the percentage as stated above. Furthermore, Steward computed the values of Kutter's n on two bases, (1) using the measured Q and computing a velocity based on the measured sections of water plus air, and (2) using the measured Q as before but using the measured velocity and the hydraulic radius as though for a net water section without air. By the first plan, he found values of n about as expected for similar surfaces in ordinary flumes. This basis can be used in computing the size of a water prism for similar construction. By the second plan he found values of n of about two thirds those holding under the first set of computations. This basis can be used in projecting actual velocities for determining the trajectory of the water at breaks in the gradient, the setting of baffle posts, or other uses where the trajectory is required. The writer has made sufficient tests of flume chutes similar to those used by Steward to confirm his deductions. The essential elements of all such tests, on both bases of computation, are given in table 2. Computations are also made on the basis of measured velocities and measured cross sections. This is the basis actually operative in the field.

(9) Another unusual feature of flume chutes, apparently paradoxical, is that algae, moss, and insect larval growth are not scoured off by the high velocities but apparently thrive on the excess of air, due to the entrainment described under (8), coupled with plenty of irrigation.

(10) In most computations for flumes at ordinary streaming velocities, the vertical depth is essentially the same as the depth measured perpendicular to the slope of the bed. Likewise, pressures are taken as though the floor had no slope. In flume chutes, the measurements of water prism, depth, and freeboard are taken perpendicular to the bed of the flume, but pressures and kinetic effects are modified to include the influence of the steep incline.

The scope of this bulletin will not permit detail treatment of the hydraulics of chute structures. Suggestions to be borne in mind are given above. Detailed treatment for various phases will be found in Bakhmeteff (*4*), Cole (*6*), Etcheverry (*8, v. 3, p. 261*), Hanna (*12*), Hinds (*13*), Husted (*16*), King (*18, p. 277, 340*), Nimmo (*22*), Steward [18], Stevens (*25*), and Stockton (*26*).

SUMMARY AND CONCLUSIONS

Detailed field measurements, evidence of existing conditions at the time of test, discussion with flume operatives, and final computations and listing of the elements of the results obtained have developed the following outstanding points:

A flume, as referred to herein is a relatively high-velocity structure, designed for the conveyance of water, with kinetic potentialities far above those usually associated with canal flow. Usually it begins and ends in a channel having a flow of much lower velocity than that prevailing in the flume itself.

The sequence of the structural elements is: (1) An inlet transition to accelerate the flow from canal to flume velocities, (2) the flume

[18] Steward, W. G., Op. cit. (see footnote 11).

proper, and (3) the outlet transition to decelerate flow from flume to canal velocities.

The flow elements entering the design of the flume proper consist of quantity, Q, usually given in the conditions of the problem; the hydraulic radius, R, computed from the formula $R=A/p$ for the tentative section being tried in computations; the velocity, V, computed from $V=Q/A$ for the tentative section; an assumed value of the frictional factor, n; and the slope, $S=s$, for uniform flow, resulting from the solution of a standard flow formula such as Kutter's or Manning's. Sometimes the slope is given and the solution is directed to the development of a sectional shape that will satisfy the other elements.

The dimensional elements given above are matters of data as given or as assumed, all of which can be attained by mere design and construction. The choice of n is a matter of judgment but must be based on empirical data. This choice can be directed by the recommendations given in this bulletin, which are based on nearly 300 actual field experiments listed in tables 2 and 3.

In the past the general tendency has been to use a value of n that might satisfy the best of conditions but was too low for those encountered in ordinary field operation.

Most flumes have served their purpose in spite of errors in the selection of n, because they were so short that a small amount of heading-up in the canal above served to increase the effective slope with a slight encroachment on the canal and flume freeboard near the upper end. Many long flumes, without the possibility of this method of increase in energy slope to an extent that would affect the complete length of flume, have lacked materially in total capacity.

The hydraulic elements and elevations of the canal at the ends of the flume and the elements of the flume proper having been developed, the flume is set in the scheme of levels to provide adequate drop in the water surface for the necessary acceleration of velocity and entry loss and also to provide for the recovery of a reasonable amount of velocity head at the outlet and a distinct loss of energy expressed as the velocity head that is not recovered.

After the dimensions of canal and intervening flume sections and their relative positions in the vertical plane have been determined, the bottom profile in terms of available velocities must be studied under the assumption that more or less debris, rolling along the channel bottom, will be present. Very few canals are free from such detritus. With slightly accelerated velocities, this debris will climb a gentle upward incline but heavy debris will lodge against a vertical upward offset in the bottom. Sugar sand will travel in dunes down a channel and will be picked up by the scurry of the eddy always present just above such an offset, and continue its way down the canal. Sharp offsets in profile are to be avoided as they are generally indicative of hydraulic losses and roughness of flow.

After flumes are built, the difference between long and short structures becomes apparent. The long flume always has some length conveying flow that is relatively uniform without regard to the amount flowing. The short flume seldom attains even approximately uniform flow conditions except by coincidence and with a discharge close to the design Q'.

The short flume and the extreme ends of a long flume, for flows other than those close to design Q', may have drop-down or back water curves, critical flow points, hydraulic jumps, and lag in the surface drop at the inlet. These may or may not develop operation troubles.

A long drop-down curve may be used in a flume with a brink outlet into a pool, for all possible stages of tail water, to save in construction by reducing the total depth as the outlet is approached. That is, the flume may be made shallower as the velocity is increased, until critical velocity at the brink is reached.

For a long or short flume, the concave surface curve, when indicating true backwater by reduction in the effective slope, always is accompanied by reduced capacity, for any given depths, compared with usual design elements.

For a short flume, the convex surface, indicating drop-down curve by increase in the effective slope, always is accompanied by increased capacity for any given depth. The shorter the flume, the greater is this increase. For a given length of flume, the upper limit of this increase takes place when water reaches the outlet end of the flume at critical depth. Further depression of the water in the canal below the outlet does not further increase the capacity of the structure.

A long flume is not improved in capacity by a drop-down curve at the outlet end. The upper end still remains the controlling factor. Wherever feasible, water should be conveyed to a chute-flume in a direct line with the axis of the chute. If it is necessary to break the constructed bottom grade of the chute, thereby increasing the gradient the bottom should be curved so gradually that the trajectory of the jet will not tend to leap clear of the bottom, causing vacuum troubles. The resulting cavitation may cause the floor of a plank chute to be crushed inward by atmospheric pressure until the jet strikes plank ends and tears off the bottom.

Extra freeboard is desirable for the first 20 or 30 feet of flume to care for rough water and lag in the surface drop while flume velocity is being developed. This is easily accomplished in concrete or wood-plank fluming. In metal flumes longer sheets may be bent to the shape of the standard flume section and the excess length divided between the two sides as extra freeboard. The stringers for the first two bents may be raised to care for this additional freeboard.

A prevalent understanding of the past, expressed both verbally and in published articles, has been to the effect that there would be no recovery of velocity head at the outlet. Partial recovery is nearly always present except where water goes over the outlet end of the flume as a brink with free fall, critical depth occurring a short distance above the brink. Recovery of from 50 to 80 percent can be relied upon.

High velocities cannot be depended on to keep a flume clear of moss and algæ growth. Most operators of irrigation systems are familiar with conduit structures with velocities up to perhaps 20 feet per second in which such growths thrive. Ordinary flume velocities should be considered wholly ineffective to reduce such growths. Shade, such as is provided by a thick row of trees, or, better still, by a definite flume cover, is usually effective in preventing certain types of such growths. Experiments should be made or advice obtained from botanists before expensive construction is undertaken.

An apparent paradox exists in the action of abrasive matter carried by a flume. Usually, it is necessary to limit the permissible high velocity in order to prevent abrasion as rough material is dragged along the bottom. High flume velocities may not prevent scour by heavy detritus, but sand and fine gravel are raised off the bottom and whirled along in the water prism. This suggests the deduction that low flume velocities (but materially higher than ordinary canal velocities) may allow scouring of the bed, while very high velocities, say 12 feet or more, may materially lessen scouring, except by heavy material that may be easily trapped before it enters the flume.

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